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ARTICLES IN THIS ISSUE

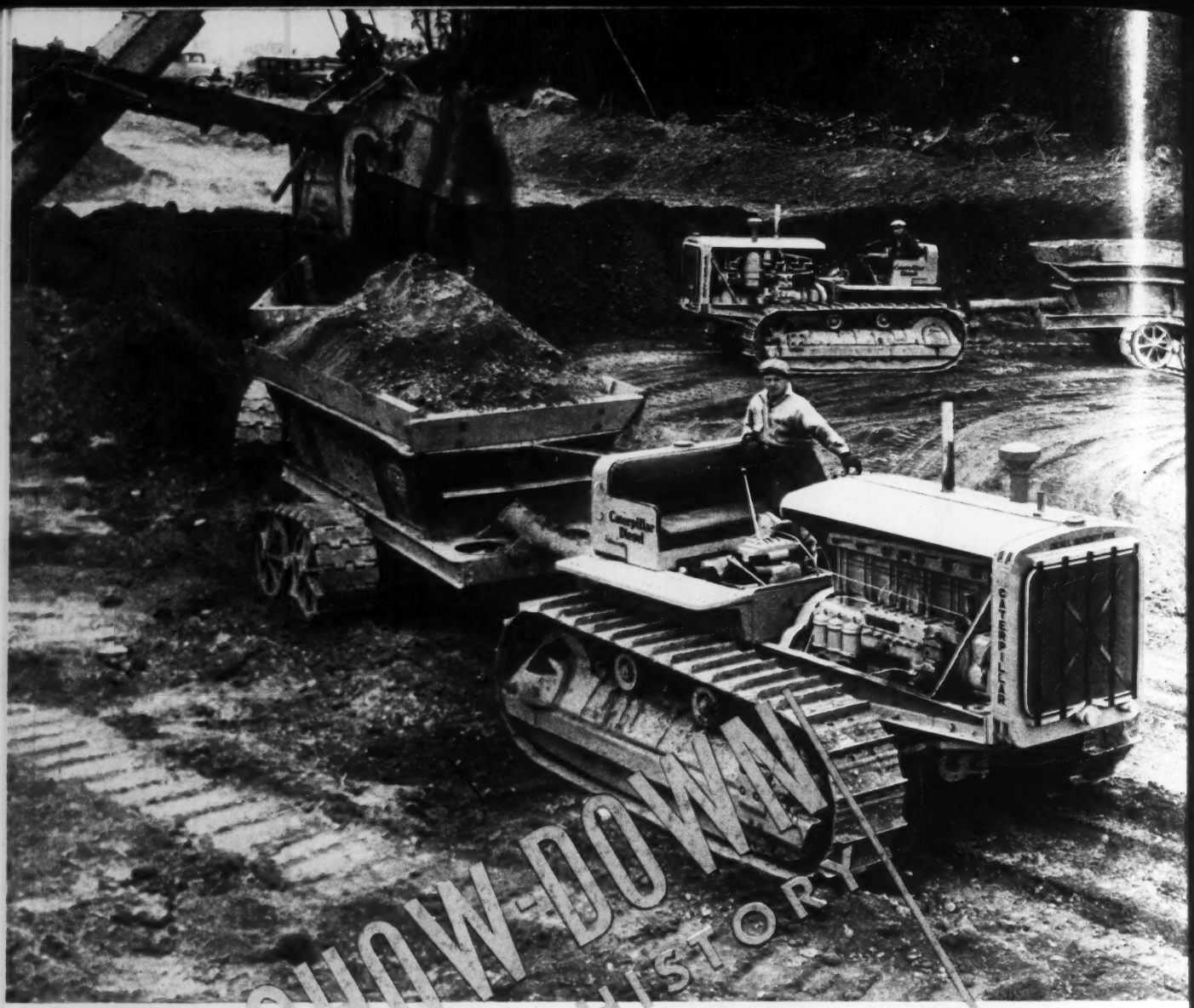
Progressive Cofferdam for a Long Low Dam

Droughts Since 1881

Composite Steel and Concrete Bridge Slabs

New Plan for Mississippi Flood Control

FOR BETTER NAVIGATION on the Allegheny River, Dam No. 3 above Pittsburgh is being built within a cellular wall cofferdam with such modern equipment as a gantry pile driver, an all-electric mixing plant, mobile form carriages and concrete vibrators.



On a road-building contract near Towson, Md., "Caterpillar" Diesel Tractors slash the costs of moving earth from cut to fill

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DIESEL

Too Much Paper, Not Too Much Plant

IN the issue of July 12, 1934, this page cited an investigation by Edwin G. Nourse and associates of the widely held notion that our productive industries are so greatly overbuilt that future demand for capital goods and services must be substantially impaired. This study was embodied in a volume entitled "America's Capacity to Produce," published by the Brookings Institution of Washington.

The net of the findings was that the margin of unutilized plant capacity was somewhat less than 20 per cent and that it had not expanded appreciably during the period from 1900 to 1930.

In commenting on this conclusion that we have had no sudden extravagant expansion of actual productive capacity, this writer wondered what might be revealed by a similar study of the financial processes incident to capital investment during that period; he hoped that future volumes of the series might shed some light on this fascinating theme.

SINCE then the Brookings Institution has issued its second volume on "America's Capacity to Consume," and now comes the third, by Harold G. Moulton, on "The Formation of Capital." Here we begin to find an explanation of the paradox that has so disturbed engineers, architects, constructors and manufacturers who must look for employment in the provision of new capital structures and equipment. For this new study helps to reconcile the contention that too small a share of the national income flows into consumptive channels with the fact that we have not, in reality, experienced any extravagant expansion of productive capacity.

The paradox arises, it appears, from our too-ready acceptance of the classical economic theory that an increase in savings must result automatically in a corresponding expansion of new plant and equipment. On this assumption rests the theory that we are immensely overbuilt in physical capacity. The fact is, however, that while the funds available for capital expansion did increase more rapidly than those which flowed into consumption, the actual installation of new plant and equipment did not increase appreciably faster than demand for the goods they produced.

The study shows that our destructive expansion occurred, not in the domain of physical capacity but in the volume of paper claims against the productivity of our existing plant. Berle and Pederson have shown that the volume of such liquid claims increased from about 18 per cent of the national wealth in 1900 to 40 per cent in 1930. The Moulton study shows that while savings available for investment ranged from $8 billions in 1923-24 to $15 billions in 1928-29, new corporate issues and realty mortgages, *for productive purposes*, remained stationary at about $5 billions. In other words, of our total savings only between one-half and one-third passed into the hands of enterprisers who used them to build new plant and equipment.

WHAT became of the rest of our monetary savings? The answer to this must be sought in the realm of financial speculation rather than of productive industry. It is indicated by the amazing increase in the volume of securities issued during that period in behalf of purely financial institutions that added nothing to the real worth or productivity of the nation but served merely to inflate the volume and market prices of the claims outstanding against the productivity of our existing plant. Thus the study leads to the conclusion that "aside from the portion that went into foreign issues, the excess savings were absorbed, dissipated, in bidding up the price of outstanding securities. Money savings were thus transferred increasingly into speculative profits rather than into productive plant and equipment."

Every producer engaged in construction and its dependent industries will find food for thought in this analysis. Unfortunately, the excessive burden imposed upon productive enterprise by this misuse of our surplus savings has been almost as effective a brake on recovery as an actual over-capacity would have been. But it is heartening to know that when more of it has been written off, as it must be, we shall see ever more clearly that industry is not in fact overbuilt. And in the light of this and kindred studies, we should be encouraged to undertake such reforms in our financial structure and practice as will foster the application of future savings to genuine productivity rather than to the deceit and destruction of financial manipulation.

Willard Chevalier

NRA CODE

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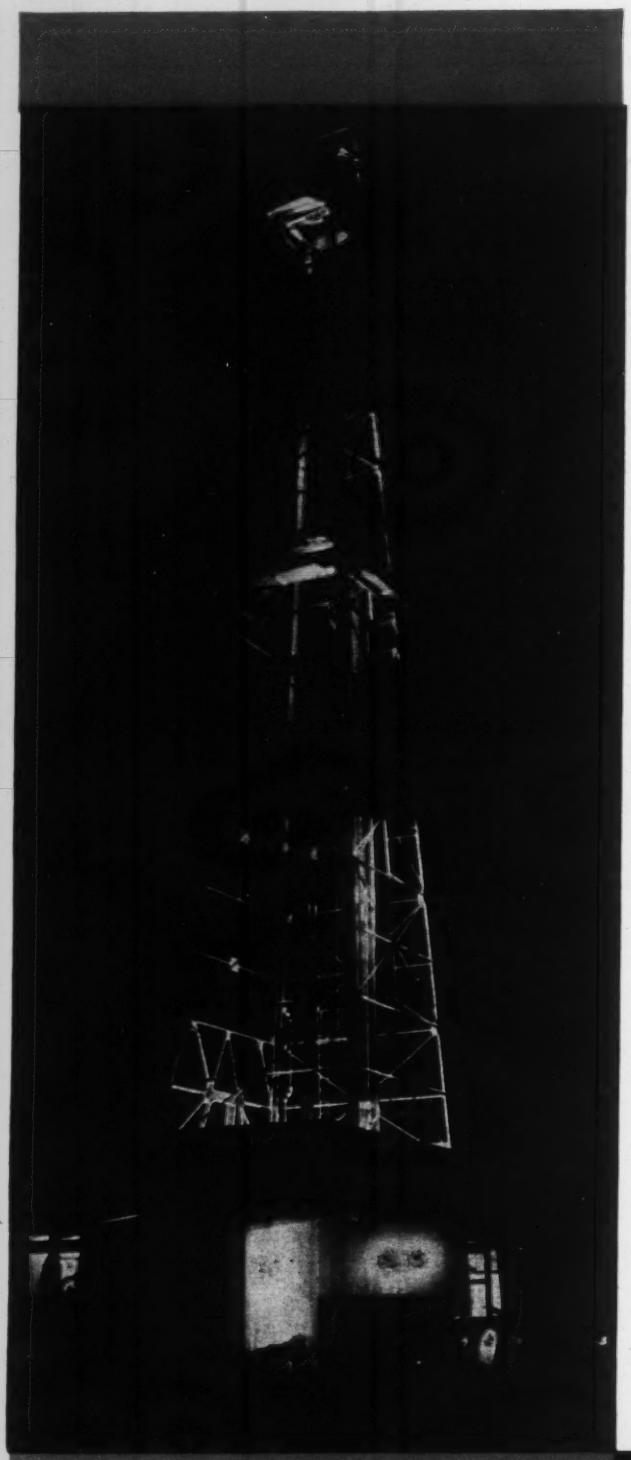
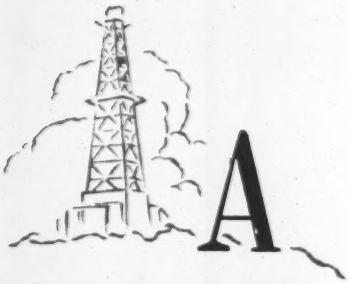
Table of Contents—See Page 23





A CONTRAST BASED ON facts

It is difficult by the hot process method of pipe manufacture to produce a product that is truly round. Furthermore, the wall thickness will vary as much as 12½ per cent. When threads are cut on this pipe, they are not of uniform depth, and a variation in the thickness of metal under the threads is a natural conclusion.

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CONTENTS FOR APRIL 4, 1935

VOLUME 114 NUMBER 14 COPYRIGHT 1935

Progressive Cofferdamming on Long Low Dam 473

Extent and Severity of Droughts Since 1881.. 478
BY LLOYD L. HARROLD

More Data on Boulder Dam Spillway Models. 480
BY D. C. McCONAUGHY

Hydraulic-Fill Shrinkage in Miami Dams..... 482
BY C. H. EIFFERT

Composite Steel and Concrete Bridge Slabs.. 484
BY SHORTRIDGE HARDESTY

Utah Court Decisions on Underground Water 487
BY C. J. ULLRICH

Prestressed Concrete Standpipe Design..... 488
BY A. P. KURANZ

Hangar Frame Built of Discarded T-Rails... 490
BY F. N. WILDISH

Light Sheet-Metal Wall for Airport Building. 491
BY GEO. D. BURR

Revised Plan for Mississippi Flood Control.. 492

Editorials 496

Current News 498

The Business Side of Construction..... 504

Prices of Construction Materials..... 506

In the News:

A BILL HAS BEEN PASSED in Arizona prohibiting the building of an obstruction across a navigable stream within the jurisdiction of the state without the consent of the governor and attorney-general. It is calculated to give Arizona a new foothold in the fight over the construction of Parker Dam.

A POTENTIAL SHORTAGE of power-generating equipment in this country is cited in the first interim report of the National Power Survey by the Federal Power Commission. Instead of a surplus of power, the commission reports that there will be a serious shortage as soon as business conditions improve.

ENGINEERS have been put to work on detailed plans for the proposed Sixth Avenue subway line, the last link in the municipal system at New York. It is hoped to get money for the new line under the relief act now pending in Congress.

CONTRACTS with private architectural firms have been signed by the PWA for two housing projects in Nashville, Tenn., one of which is for negroes.

A REGISTRATION LAW for engineers and land surveyors has been passed in Utah. With Washington and New Mexico also recently added, this brings the list up to 30 states.

APPOINTMENT by the PWA of a project engineer for the Triborough Bridge and the Midtown Hudson Tunnel, who is not licensed to practice engineering in New York State, has been protested by the state board of engineering examiners.

CONSTRUCTION of Knoxville's municipal power-distribution system has been stopped by a temporary injunction. The city is charged with entering into unlawful competition with the utility company serving its citizens.

SILICOSIS and other occupational diseases are now covered by the compensation laws of New York and North Carolina. New York, in a bill signed last week, includes "all occupational diseases"; North Carolina's new act definitely names 25 to be included, and in addition provides for inspection of hazardous occupations and for definite medical diagnosis of occupational diseases. New York has also revised sections of its compensation law relating to medical treatment of injuries, correcting a number of flagrant abuses.

In This Issue:

THE LOCK AND DAM PROGRAM to improve the Allegheny River for navigation at and above Pittsburgh has advanced another step with the virtual completion of Dam No. 3. Dam No. 2 was completed last year. Construction of Dam No. 3 involved progressive cofferdamming and novelties in bulkheading and piledriving.

DUST STORMS AND DROUGHTS have sent engineers to the data files for background information on water supply and precipitation. Considerable aid is afforded by an article in this issue on the frequency, extent and severity of droughts since 1881.

SPILLWAY DESIGN for Boulder Dam is reviewed by D. C. McConaughy, of the Bureau of Reclamation, in the light of some criticisms published in *Engineering News-Record* by Swiss engineers who had studied the design by means of models.

SHRINKAGE RECORDS for some of the hydraulic-fill dams of the Miami Conservancy District are brought up to date.

LIGHT-WEIGHT SLABS of concrete-filled steel-grid type are being used on a number of bridges. The necessary design calculations, when considering these slabs to be of composite construction, are outlined in connection with two installations in New Jersey and a test program at Columbia Universiy.

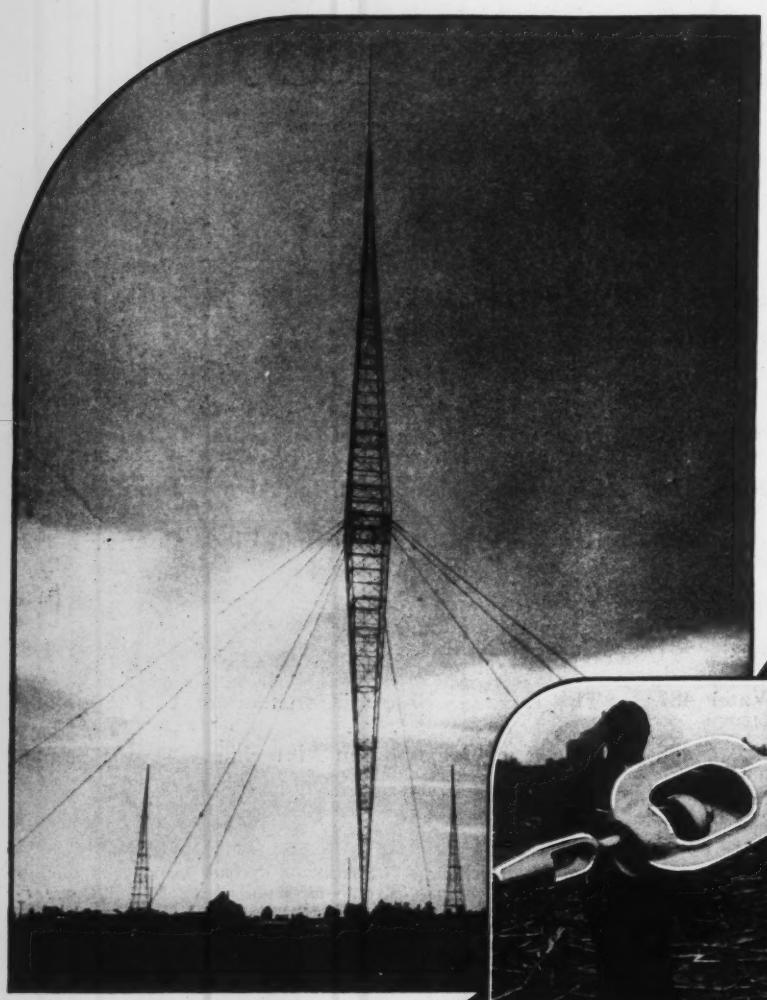
WITH THE GROWING IMPORTANCE of underground waters as sources of public supply, the legal difficulties attendant on their regulation continue to grow. The situation in Utah, outlined in this issue, is therefore not only important in itself but of wide significance.

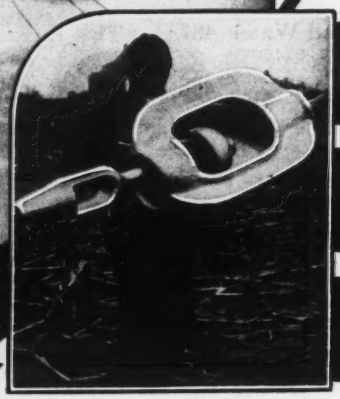
INTERESTING SMALL STRUCTURES of engineering importance described in this issue include a two-story office building with steel-plate exterior walls, a water tank of prestressed concrete, and an airplane hangar with an arch roof of old T-rail ribs.

Coming Articles:

NEXT WEEK the leading article will be devoted to the remarkable construction procedure used in driving the new 36-ft.-diameter tunnel for the Pennsylvania Railroad in Baltimore.

THE ANNUAL WATERWORKS NUMBER will be published on May 2. It will comprise a group of articles prepared by acknowledged authorities, who will discuss various phases of the constantly evolving character of the water-service problem. Source of supply, purification, distribution and administration will all be covered.





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ENGINEERING NEWS-RECORD

April 4, 1935

FIG. 1—NAVIGATION DAM, No. 3, Allegheny River, showing progression of construction within the cofferdam, which also is progressively extended as construction advances.

Progressive Cofferdamming for Long Low Dam

Allegheny River navigation dam carried across river by advancing cofferdam in five steps and tearing down behind as concreting of each dam section is completed—Novelties worked out in bulkheading and piledriving—Lessons in vibrating concrete

DAM CONSTRUCTION for the new locks and dam No. 3 of the Allegheny River navigation works developed a number of experiences with plant and methods that are uncommon in the routine of such standard construction. Construction of the dam was carried out progressively from one shore by building the cofferdam in five successive sections and taking out the section behind as soon as the structure within it was completed. A gantry carriage riding on the cofferdam walls and mounting a shuttling piledriver drove the many hundreds of piles for the dam foundation with more than usual dispatch and accuracy. The use of internal vibrators in placing the concrete indicated a number of factors having bearing on the general conduct of this method of consolidation. Other construction procedure did not depart notably from ordinary practices for each kind of operation.

Dam structure

Dam No. 3 is a fixed navigation dam of concrete of conventional gravity section carried on piles for 1,358 ft. and going down to shallow rock for 78 ft. at the lock end. Fig. 2 is a section of the dam on piles; the section on rock is the same except that the concrete is carried deeper to rock, giving a heavier base and apron. All transverse dimensions are the same throughout the dam length. This is true even of the part of the dam that for about 400 ft. crosses Fourteen-Mile Island, a low sandbar-type island that divides the river into two channels. A dredge cut about 400 ft. wide was carried across the island to the depth of the top of the apron, and the dam was built in this cut by the regular cofferdam method.

Referring to Fig. 2, a continuous apron wall 5 ft. high forms a stilling pool, and back of this is a row of baffle piers 5 ft. wide and high, spaced 5 ft. apart to break up the current. Two cutoff walls of steel sheetpiles will be noted, one driven to rock under the heel of the dam, and the other under the toewall of the apron but driven only to 15 ft. depth. The foundation piles were spaced as shown, with the piles 4 ft. apart in the rows. They were required to be driven to refusal. The cutoff length of the longest piles was about 30 ft., and the cutoff length averaged generally about 17 ft. for the dam.

A portion of one wall shown by Fig. 3 indicates in general the cofferdam wall

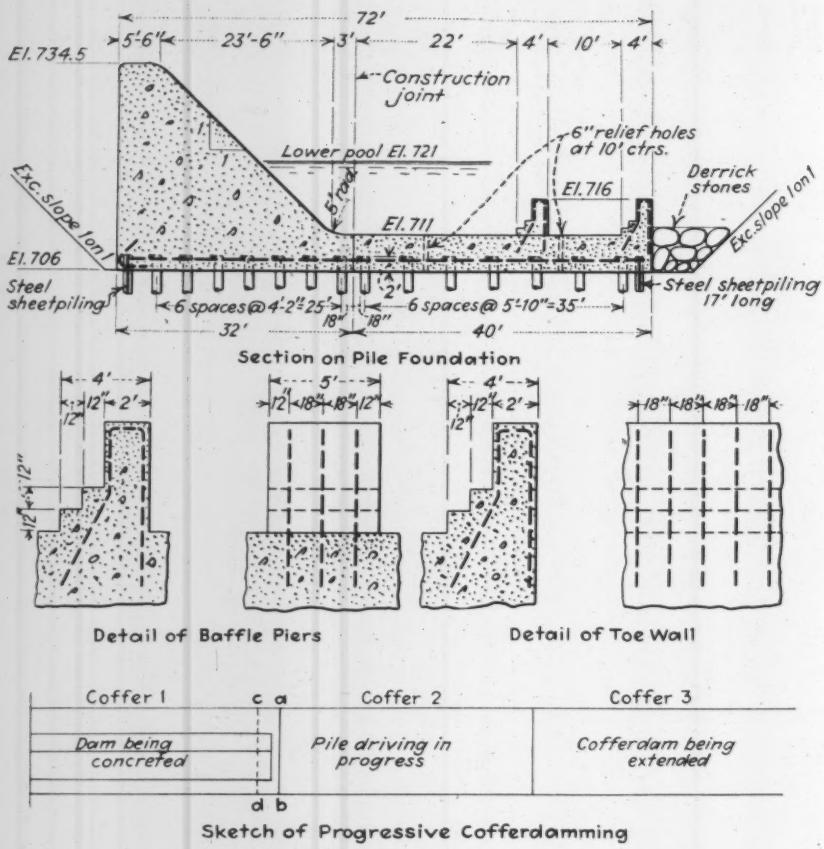


FIG. 2—STANDARD SECTION and pier and toewall details of Allegheny River navigation dam No. 3, showing also progression of coffers.

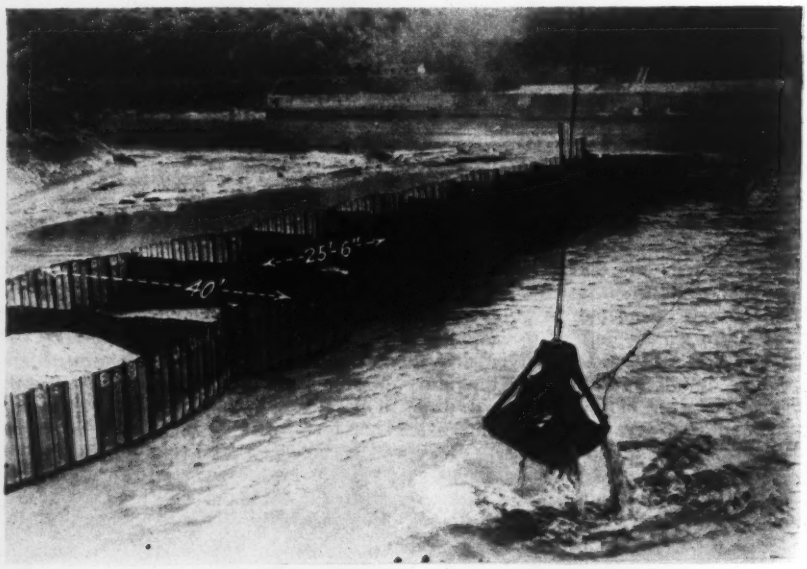
structure of steel sheetpile cells, with arched side shells and straight partitions. In the outside (river side) shells the piles were 50 ft. long driven to rock, and for the inside shells and partitions they were 34 ft. long driven to top elevation with the piles to rock. This gave a flat surface to the top of the wall for tracks for handling material and particularly for the special piledriver carriage described later, and for the same reason the upstream and downstream parallel walls of the coffer were constructed to the same elevation as shown by several of the illustrations. The sheetpiles used weighed 30.7 lb. per lin.-ft. and 2.3 lb. per sq.ft. of shell.

Little that was novel was introduced in the cell construction proper. Using a floating timber templet, the Y-piles at the corners were driven first, after which threading and driving the shell piles proceeded rapidly. Two driving rigs were used; each was a stiff-leg derrick mounted on two pontoons, a rear pontoon 60x18 ft. carrying the stiff-legs, counterweights and steam hoisting unit, and a smaller forward pontoon carrying the mast. The derricks were of usual construction, but a useful variation in driving rig was introduced. No regular leads were used; instead an H-beam was suspended from the derrick, and the double-acting hammer having special angle clamps rode this beam. In operation the bottom of the H-beam was tied to the lifting hole in the pile last driven, and a strain was taken on the beam by the derrick, holding it vertical. The hammer riding the beam was thus in position for driving the new pile. Exceptionally fast change from pile to pile was found possible with this rig. Filling the cells with gravel from the river bed was a simple clamshell operation as indicated by Fig. 3.

The principal complication in the cofferdam construction was introduced by installing it in successive sections. Five sections of coffer, about 300 ft. long, were built for the 1,436 ft. of dam length, working from the right-hand bank toward the locks built on the left-hand bank by a previous contract. As the concreting in each coffer was completed, the cofferdam walls were removed, giving free passage for the river through gaps left in the upper part of the concrete structure. A diagram (Fig. 2) indicates procedure. Coffer 1 was completed with its end wall *ab*; it was pumped out, the piles were driven, and concreting followed. Meanwhile coffer 2 had been built, and excavating and pile-driving were begun. When concreting in coffer 1 had proceeded to end wall *ab*, a bulkhead *cd* was built, end wall *ab* was torn out, concreting was continued into coffer 2, the sheeting was pulled in the side walls of coffer 1, and the river was turned through. In exactly the same way construction proceeded through succeeding coffers to the end of the dam. The objective of the plan was, of course, to keep successive operations in simultaneous progress so far as possible and so reduce the construction period, and also to reduce the amount of material held in cofferdam structure beyond the time that such structure was actually needed to protect the work. The sheeting was thus used repeatedly.

All of the several operations recounted are illustrated by the views Figs. 5-7; the three calling for particular

FIG. 3—CELLULAR WALL for cofferdam for navigation dam No. 3, consisting of steel sheetpiles. River sand was used for filling.



notice are the cross-bulkhead (*cd* in Fig. 2) construction, the piledriving and the concreting in which internal vibrators were used.

Bulkhead construction

Referring again to Fig. 2, to remove coffer 1 in which concreting had been completed and to continue the concreting into coffer 2 a bulkhead was, as previously said, required at some point as *cd* back of the original end wall *ab* of coffer 1. To be effective, the cutoff had to provide against seepage under the completed dam as well as against inflow from the river above. The construction adopted is shown by Fig. 4.

To prevent leakage underneath the completed portion of the work, a row of steel sheetpiles was driven to rock across the dam between two crosspiles placed in the heel and toe cutoff walls (Fig. 2) and extended beyond these walls enough to clear the concrete. This cross-wall was set and driven during the regular piledriving and was sealed 2 ft. into the concrete foundation. Naturally this steel-pile seal is not salvageable. To form strong connections and a tight seal between the concrete and the steel-pile bulkhead, an interlock was embedded in the mass concrete (detail *A*, Fig. 4).

The bulkhead over the finished work was cellular, except at the break of the dam, where wales were placed along the break of the dam and horizontally, and 1-in. tierods were inserted every 4 ft. The river side of the cells, up and downstream between the finished work and the finished cofferdam, were driven to rock. The parts of the cells extending over the finished work rested on the concrete. The bulkhead was sealed at the concrete by very carefully stuffing straw into any crack between the piling and the concrete, and then a layer of fine sand and clay was placed in the cells over the concrete to a depth of about 1 ft. before filling all cells with sand and gravel.

Piledriving

About 4,000 wood-foundation piles had to be driven inside the cofferdam.

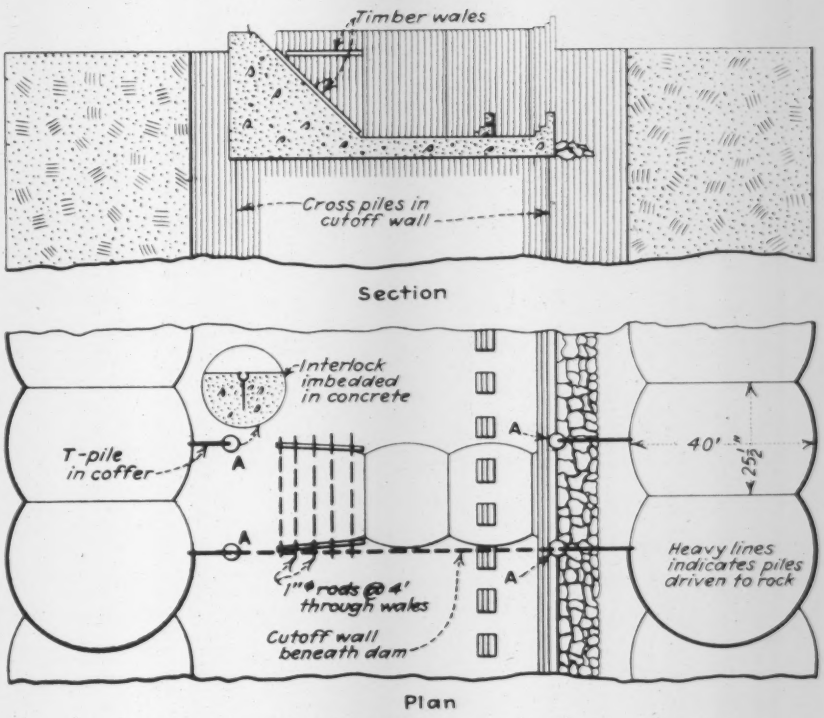


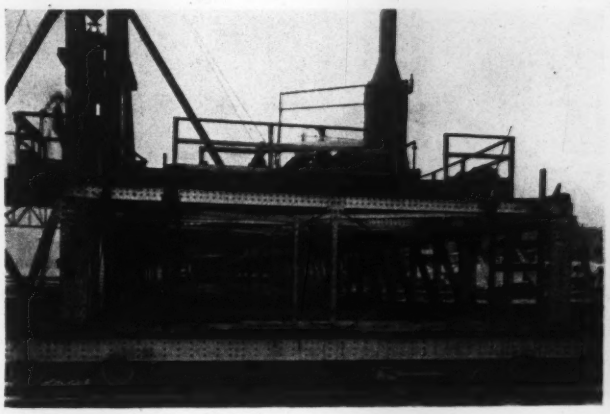
FIG. 4—BULKHEAD CONSTRUCTION to permit extension of work into coffer ahead and the removal of the coffer behind.

These piles and the steel piles for the toe and heel and the cross cutoff walls underneath the dam at the bulkheads described above (about 3,000 lin.ft. of sheeting) were driven by a special rig designed for this work by the contractor's engineers (Fig. 5).

As stated before, the cofferdam walls were well built and were capable of supporting heavy equipment. The piledriving rig consisted of a traveling gantry of 107-ft. span, made up of two trusses. The gantry was self-propelling and rode on tracks that were carried by heavy ties set on the gravel fill of the cofferdam pockets. On top of the gantry was a traveling rig, which carried the piledriving leads, set at right angles to the gantry, and a jib boom. This rig, steam-operated, did all of the driving of the foundation, cutoff and bulkhead piles. A single-acting heavy steam hammer was used for the foundation piles, and a double-acting lighter hammer was used for the steel sheeting. The small jib crane on top of the rig handled the piles into place. By the use of this rig the driving was done very rapidly, and naturally an excellent alignment could be secured as the rig could be spotted at any desired point and held there. Several views show the operation of the piledriving equipment. On special runs the rig drove 110 piles in a day, but the average was about 85 piles.

Concreting was done in three lifts: first the apron and baffle piers and

FIG. 5—PILEDRIVER GANTRY viewed in end elevation (left) and in side elevation (right). The rig drove an average of 85 piles per day.



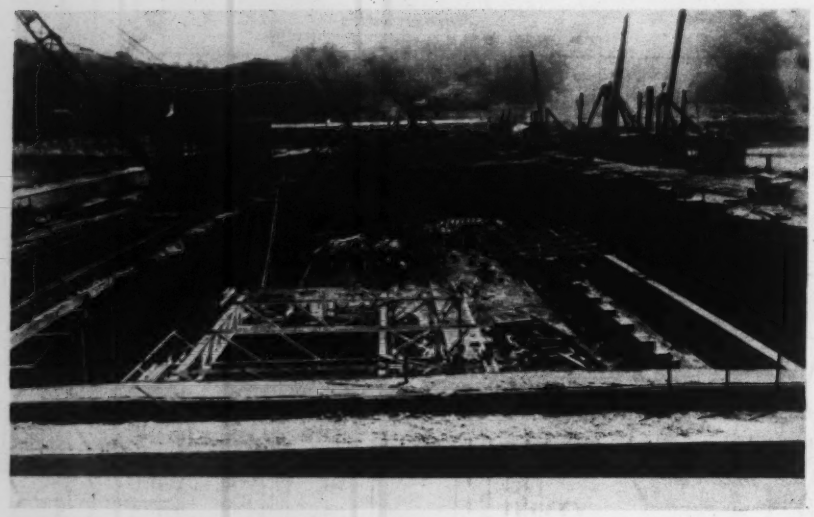


FIG. 6—FIRST COFFERDAM SECTION, showing beginning of concrete work on apron and dam base.

foundation of the dam itself (Fig. 6); second, half the height of the dam; and third, the final topping out of the dam (Fig. 7).

Concreting forms and equipment

Wood forms were used to place a 6-ft. thickness of concrete for the apron and for the bottom lift of the dam. Two steel forms placed the remaining concrete in the dam in two lifts, and a third set of steel forms cast the baffle wall and denticulated piers of the apron. A gantry carrying forms for both baffles traveled on rails laid on the apron. The steel forms for the two lifts of the dam proper also traveled on steel rails, laid on the next lower lift at the forward end and on the completed lift at the rear end. The lower lift of steel form, which is 40 ft. long, required the largest monolithic concrete placement on the work, 385 cu.yd.

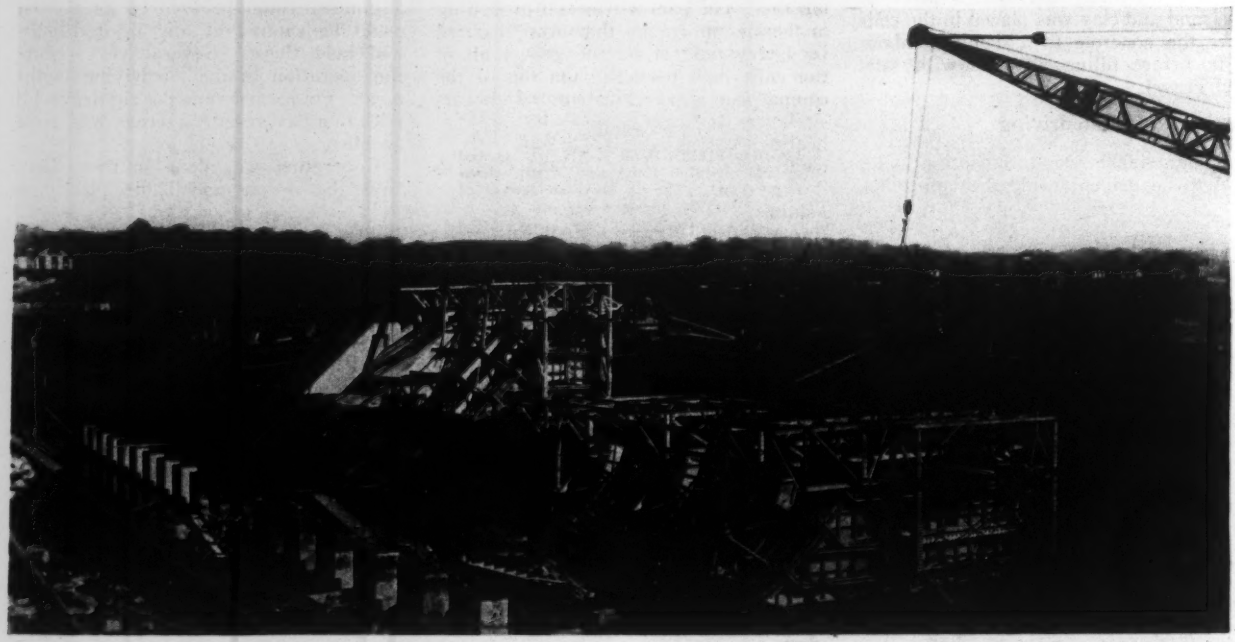
For the first 800 ft. of dam, the steel form for the upper lift was only 20 ft. long, permitting this lift to be cast in alternate 20-ft. blocks with 20-ft. openings between them. These openings, aggregating 400 ft. in length, passed the flow of the river during the construction of the last section.

Concrete for the dam was batched and mixed by a floating plant that moved along the cofferdam as the work advanced. The concrete mixture was based on 1:1.72:5.76 volume proportions and included three sizes of coarse aggregate of maximum 6-in., 4-in. and 2-in. size. Stated in dry weights for a 1-yd. batch, the proportions of the concrete ingredients were 825 lb. of sand, 730 lb. of small gravel, 730 lb. of medium gravel, 1,200 lb. of large gravel and 435 lb. of cement. These weights had to be adjusted for the larger batch mixed by the 1½-yd. mixers of the floating plant.

A modern, electrically operated batching and mixing plant was mounted on a pair of spliced steel hulls, which also carried a steam whirley with a 75-ft. boom that charged the overhead aggregate bins from supply barges with a 1-yd. clamshell bucket. Bulk cement was transferred to a 100-bbl. overhead bin by a Fuller-Kinyon conveying system from a covered barge of 2,300-bbl. capacity. One set of batching equipment fed two 1½-yd. mixers. The proportioning equipment (of 2-yd. size) was operated by one man, with pushbutton control of all functions except aggregate batching, which the operator took care of with manually thrown levers. An automatic weighing batcher of screw-feed type, an automatic water-weighing and measuring tank and a springless dial scale for four classes of aggregate measured the concrete ingredients; mixer batchmeters were set for a 1½-min. mixing period.

Mixed batches were discharged into 1½-yd. roller-gate buckets, which were handled into the forms by a steam whirley with an 85-ft. boom traveling on the cofferdam. These buckets were built up from standard 1-yd. buckets by adding an 18-in. extension, to give the required greater capacity, while keeping the diameter small enough to go inside the crest form on the dam. The dam

FIG. 7—FORMS for building dam section in two lifts travel on trackways.



and abutment called for 46,000 cu.yd. of concrete. All concrete was vibrated internally with electric spud vibrators.

Concrete vibration

The particular feature of the concreting was the compaction by internal vibrators. The specifications called for the use of "special pneumatic or electric vibrating or tamping machines or devices." The contractor elected to use electric internal vibrators, specifically Vibro-Spades, with a minimum frequency of 3,600 r.p.m. As all concrete on this project has been placed with the aid of vibrators, no comparative data are available, and the following conclusions as obtained from the engineers on the work are only of a general nature.

A maximum water content of 6.5 gal. per sack was specified, with a minimum 2-in., maximum 7-in., slump. Taking advantage of the maximum water content, a water-cement ratio of 0.866 would result. However, an average slump of 1¾ in. with an average water cement ratio of 0.733 has been maintained. To date, with 75 per cent of the total concrete placed, the 28-day cylinder tests have averaged 4,200 lb. per sq.in. Considering the reduction effected in water-cement ratio, it may be assumed that an increase of approximately 1,000 lb. per sq.in. for the 28-day cylinder test has been gained over the results that might have been expected from unvibrated concrete. Tests of 6-in. cores, drilled from completed monoliths, have confirmed the test-cylinder data. The vibrators have been used to break down the pile of concrete as it was discharged from the bucket and also along the face of the form. As the result of vibrating near the face of the form, the exposed surfaces of the concrete have been unusually free from honeycombing. However, when steel forms were used, numerous small pits or pock marks on the surface adjacent to the forms were noticeable. It is assumed that this condition was the result of the segregation of minute air pockets, the larger voids forming against the steel form through which they could not escape.

Excessive vibration was found to segregate and draw to the surface an excess of paste and water. Vibration of wet concrete produced like results. Also it was found most advantageous to use two vibrators in each pour, and although very few breakdowns were experienced, the contractor maintained a third vibrator at the form as a stand-by. This precaution was particularly advisable when pouring dry concrete, as the contractor found that 1½-cu-yd. batches of 2-in. slump concrete were slow and expensive to place without the aid of vibrators.

Even with the use of vibrators, the contractor used a concrete gang of six to eight men. These men used shovels to scatter out the concrete as it was dumped from the bucket. Dependence cannot be placed on the vibrators to do this, as excessive vibration with consequent segregation would result. The vibrator was left in one place for only about 10 sec.—just long enough to sink in the mass and come out again slowly in order that the concrete would fill in behind the vibrator as it was being removed. The use of vibrators in no way removed the necessity of having a competent, experienced foreman in charge of placing the concrete.

Allegheny River Dam 3 is being built with a PWA allotment by the Corps of Engineers, U. S. Army, from plans prepared by the district office, Pittsburgh. Maj. W. D. Styer, district engineer, is in general charge of design and construction. Operations at the site are directed by C. H. Wagner, resident engineer. For Seeds & Derham, of Philadelphia, the contractors, William Lynch is general superintendent at the job, Samuel D. Ashworth is superintendent, and P. C. Olin is field engineer.

Electric Heating Units Help Unload Liquid-Asphalt Tank Cars

TO COPE WITH high freight rates, handling and heating costs, a prominent manufacturer of high-grade roofing material has found it economical to use electric heat to assist in unloading Mexican asphalt when received in tank cars in liquid form. Mexican asphalt, which has a melting point of 225 deg. F., is loaded into the tank car at a temperature of 450 to 600 deg. F., but during transit a portion of the asphalt cools, settles to the bottom and forms a hard layer. The valve through which the asphalt is pumped off is located in the bottom of the car, and this cold layer makes it impossible to start a flow through the value.

Although all tank cars used for this purpose are equipped with steam coils, it is a slow process to get this valve working unless superheated steam is used. Several method of accomplishing this have been worked out, but most of them have the disadvantage of being very expensive from both the installation and operation standpoints. One company would have required an installation cost of $14,000 for an oil-heating installation, which did not include the cost of buildings and erection. To get around the difficulty of unloading liquid asphalt, some manufacturers have Mexican asphalt shipped in cold form, which is also expensive from the standpoint of high freight rates, handling and heating costs.

According to the General Electric Co., the roofing manufacturer has solved the problem in a very practical manner by means of localized electric heat using three 4-kw. heating units, similar to those designed for oil and salt baths. Each unit is mounted on the end of a 10-ft. 1-in. conduit, on the opposite end of which is a heavy-duty receptacle. Another set of receptacles was arranged on the side of the pump house. Connections were made between these two points with rubber-covered, two-conductor cable, mounting heavy-duty plugs.

When a car of asphalt arrives, the manhole cover is removed, and the three 4-kw. units are inserted through the opening and placed as close to the valve opening as possible. Steam connections are also made to melt any asphalt that might cling to the steam coils. In an hour or two the asphalt surrounding the valve opening is thoroughly melted, and pumping can proceed. The electric units remain in operation during the time the material is being pumped from the car, to break up any lumps that might obstruct the flow of the asphalt. This operation usually requires from 8 to 12 hours.

The function of these heating units is merely to keep the valve open. They are not intended to heat all the asphalt in the tank car.

The complete equipment costs less than $1,000 installed, and is very economical from an operating standpoint. The manufacturer estimates that this equipment saves approximately $100 per car of material unloaded.

Garbage-Grinding Plant Installed at St. Louis

A new $25,000 garbage-grinding plant has recently been put into service by the city of St. Louis, Mo. The garbage, after being ground, is deposited into the Mill Creek sewer, and from there goes into the Mississippi River. The plant is equipped with two grinding machines, one of 20 tons per hour capacity, and the other 10 tons per hour. The plant capacity is rated at 300 tons per day. At present about 70 tons per day are being collected. Frank J. McDevitt, director of streets and sewers, states that the plant has been in operation for some weeks, and daily inspections indicate that there is no accumulation whatever in the sewers. The city plans, at a later date, to open two additional grinding plants, one in the northern and the other in the southern section of the city.

Frequency, Extent, and Severity of Droughts Since 1881

By Lloyd L. Harrold,
Assistant Engineer, U. S. Geological Survey
Washington, D. C.

THE OCCURRENCE of two major and two secondary droughts in the last five years (1930-34) has directed much attention, especially by engineers, to approaching limits of water supplies as related to various human activities. In this it is important to know not only the frequency with which droughts are likely to occur but also something of their extent and severity.

A drought may be said to prevail when natural vegetable growth and established activities suffer as a result of insufficient water. In this connection abnormal distribution of precipitation throughout the year may be as important a factor in causing a drought as shortage in the total annual amount. Temperature and wind taoin taointaoin portant part, especially in relation to the damage done. Although deficient precipitation is the prime cause of droughts, it is not possible to set a definite limit of total annual precipitation for any region below which droughts may be produced. In general, however, serious drought effects do not result unless the annual precipitation is as low as 85 per cent of the mean—that is, an annual deficiency of 15 per cent or more. This limit has been used in drought studies in defining a "drought year" and will serve for many comparisons.

While the information in regard to past droughts is wholly inadequate to make complete comparisons, precipitation records of the U. S. Weather Bureau, which are on a comparable basis by state averages since 1880, indicate that in the humid states annual fluctuations are large in absolute amount but are comparatively small in per cent of the mean, seldom exceeding 15 per cent. In the arid states, however, fluctuations in per cent of mean are wider, and annual deficiencies ranging from 20 to 45 per cent are not unusual. Any comparisons based on state annuals for the arid states may be misleading because of the wide range of precipitation in various parts of the individual states. In the humid and semi-arid states, except Texas, the state distribution is more uniform.

The number of years in the period 1881 to 1934 when the deficiency of precipitation in per cent of the 50-year mean was 15, 20, 25 or 30 per cent or more is given by states in Table 1, which also gives the six lowest years for each state during the same period, arranged in order of magnitude of de-

TABLE I.—SIX YEARS OF LOWEST PRECIPITATION, 1881-1934

States	50-Year Mean 1881-1930 Inches	1			2			3			4			5			6			No. Years Precip. Did Not Exceed Following Percentages			
		Date	Precipitation		Date	Precipitation		Date	Precipitation		Date	Precipitation		Date	Precipitation		Date	Precipitation					
			Inches	% of M.		Inches	% of M.		Inches	% of M.		Inches	% of M.		Inches	% of M.		Inches	% of M.	85	80	75	70
Humid																							
Me.	41.20	1897	30.60	74	1914	32.98	80	1905	33.23	81	1921	33.55	81	1924	34.37	83	1910	34.51	84	8	2	1	0
N. H.	39.52	1908	30.76	78	1894	31.38	79	1899	31.66	80	1897	32.33	82	1914	32.58	82	1910	33.41	85	7	3	0	0
Vt.	37.03	1896	28.78	78	1882	29.67	80	1914	30.16	81	1908	30.24	82	1911	30.54	82	1897	31.18	84	6	2	0	0
Mass.	42.39	1897	32.43	77	1930	33.84	80	1910	34.96	82	1883	35.77	84	1908	36.04	85	1924	36.08	85	6	2	0	0
R. I.	44.07	1930	32.83	74	1897	34.60	79	1918	36.57	83	1925	37.20	84	1896	37.25	85	1924	37.46	85	8	2	1	0
Conn.	44.96	1930	32.02	71	1924	34.84	77	1897	37.50	83	1894	38.01	85	1910	38.33	85	1914	38.44	85	5	2	1	0
N. Y.	39.01	1930	32.18	82	1908	33.54	86	1895	33.89	87	1899	34.18	88	1934	34.97	90	1884	35.23	90	1	0	0	0
N. J.	45.89	1930	35.28	77	1931	37.07	81	1895	37.29	81	1918	37.65	82	1885	37.87	83	1921	38.16	83	8	1	0	0
Penn.	42.47	1930	28.82	68	1895	33.51	79	1922	34.88	82	1900	37.31	88	1931	37.36	88	1909	37.38	88	3	2	1	1
Md. & Del.	41.89	1930	23.78	57	1895	34.47	82	1925	34.91	83	1914	35.97	86	1904	36.49	87	1900	36.66	88	3	1	1	1
Va.	42.18	1930	24.99	59	1925	32.53	77	1921	34.94	83	1881	35.40	84	1894	35.76	85	1904	36.11	86	5	2	1	1
N. C.	50.44	1925	37.33	74	1930	38.04	75	1933	39.28	78	1911	42.65	85	1921	42.92	85	1904	43.27	86	5	3	2	0
S. C.	48.21	1925	35.82	74	1933	35.95	75	1931	37.17	77	1911	39.80	82	1930	40.15	83	1904	40.98	85	6	3	2	0
Tenn.	50.30	1930	39.80	79	1925	40.50	81	1904	40.74	81	1894	42.65	85	1895	43.10	86	1931	43.24	86	4	1	0	0
Ky.	45.81	1930	27.86	61	1894	34.81	76	1904	35.10	77	1901	35.65	78	1889	35.97	79	1934	37.00	81	7	5	1	1
W. Va.	43.83	1930	25.43	58	1895	32.82	75	1904	33.33	76	1894	34.52	79	1887	35.33	81	1885	37.23	85	6	4	2	1
Ohio	38.44	1934	26.60	69	1930	27.00	70	1895	28.46	74	1894	29.75	77	1901	32.36	84	1900	32.82	85	6	4	3	2
Mich.	31.29	1930	22.62	72	1934	25.46	81	1925	25.51	82	1910	25.69	82	1889	26.86	86	1895	26.90	86	4	1	1	0
Ind.	40.17	1934	29.70	74	1930	29.70	74	1901	30.56	76	1895	30.99	77	1914	31.54	79	1894	32.21	80	7	5	2	0
Ill.	37.26	1901	25.72	69	1930	27.89	75	1894	28.89	78	1914	28.99	78	1895	31.89	86	1910	32.09	86	4	4	2	1
Ark.	48.97	1901	35.27	72	1924	37.03	76	1885	37.55	77	1896	37.72	77	1917	40.72	83	1899	41.49	85	6	4	1	0
Mo.	40.17	1901	25.28	63	1930	31.27	78	1917	32.06	80	1887	32.77	82	1894	33.18	83	1934	34.19	85	6	3	1	1
Iowa	31.48	1910	19.87	63	1894	21.94	70	1901	24.41	78	1886	24.71	78	1933	24.94	79	1889	24.95	79	9	6	2	2
Wis.	31.63	1910	21.41	68	1895	22.45	71	1930	25.08	79	1932	25.37	80	1891	26.12	83	1901	26.34	83	8	4	2	1
Minn.	25.91	1910	14.73	57	1889	19.08	74	1923	19.81	76	1934	20.30	78	1929	20.56	79	1933	20.97	81	9	5	2	1
Fla.	52.94	1927	40.71	77	1917	41.36	78	1931	43.97	83	1883	45.17	85	1921	45.24	85	1895	45.50	86	5	2	0	0
Ga.	50.48	1931	36.75	73	1904	37.17	74	1893	40.29	80	1927	40.65	81	1921	40.94	81	1925	41.00	81	7	3	2	0
Ala.	53.11	1904	39.21	74	1931	42.78	81	1889	43.30	82	1914	44.90	85	1925	45.20	85	1910	45.20	85	8	1	1	0
Miss.	53.32	1889	38.31	72	1924	40.06	75	1904	41.48	78	1896	43.13	81	1899	44.52	83	1887	44.57	84	7	3	2	0
La.	55.64	1924	38.34	69	1889	40.25	72	1917	40.27	72	1899	42.35	76	1904	43.83	79	1896	46.56	84	8	5	3	1
Semi-arid																							
Okla.	32.63	1910	18.92	58	1891	22.31	68	1917	22.39	69	1901	22.78	70	1888	23.40	72	1896	23.78	73	14	10	6	4
Kan.	27.48	1917	19.60	71	1910	19.67	72	1934	19.92	72	1893	20.25	74	1894	20.72	75	1890	21.16	77	12	7	5	0
Neb.	23.50	1894	13.30	57	1934	14.40	61	1893	16.80	71	1910	17.03	72	1890	17.18	73	1895	18.75	80	9	6	5	2
S. D.	20.77	1934	13.18	63	1931	14.66	71	1894	15.30	74	1933	15.31	74	1910	15.49	75	1925	15.90	77	12	9	5	1
N. D.	17.70	1934	9.49	54	1917	10.92	62	1889	11.30	64	1910	12.19	69	1933	13.34	75	1929	14.32	81	10	5	5	4
Texas	30.84	1917	16.21	53	1893	20.47	67	1910	21.46	70	1901	22.23	72	1909	23.45	76	1924	23.50	76	11	7	4	3
Arid																							
N.M.	14.49	1910	9.46	65	1917	9.49	65	1892	9.51	66	1934	9.96	69	1902	9.97	69	1894	10.47	72	12	12	9	5
Col.	16.79	1934	10.93	65	1890	11.97	71	1888	12.00	71	1893	12.89	77	1889	13.73	82	1924	13.75	82	12	4	3	1
Wyo.	14.05	1886	7.16	51	1887	8.80	62	1902	9.81	70	1919	10.46	74	1893	10.67	76	1889	10.81	77	13	8	4	3
Mont.	15.21	1889	9.24	61	1931	10.09	66	1890	10.78	71	1904	11.04	73	1919	11.14	73	1934	11.25	74	10	7	6	2
Ida.	17.17	1888	10.08	59	1889	12.05	70	1924	12.46	73	1929	13.06	76	1928	13.60	79	1895	13.63	79	9	6	3	2
Nev.	9.01	1928	4.87	54	1887	5.18	57	1924	5.49	61	1910	5.53	61	1888	5.76	64	1929	5.83	65	16	16	11	7
Utah.	12.67	1887	6.55	52	1883	7.72	61	1900	8.38	66	1890	9.00	71	1902	9.17	72	1934	9.49	75	18	11	6	3
Ariz.	13.10	1885	7.12	54	1900	8.17	62	1903	8.43	64	1924	8.68	66	1899	8.70	66	1904	9.24	71	15	13	11	5
Calif.	23.53	1898	10.35	44	1923	14.13	60	1887	14.63	62	1895	14.85	63	1929	15.00	64	1932	15.60	66	19	15	10	7
Oreg. east.	13.81	1908	9.32	68	1898	9.72	70	1924	10.42	76	1918	10.53	76	1905	10.80	78	1928	11.21	81	10	5	2	2
Oreg. west.	54.22	1930	37.11	68	1929	38.66	71	1918	43.28	80	1905	44.11	81	1892	44.25	82	1923	44.89	83	9	3	2	1
Wash. east.	16.89	1929	10.19	60	1890	12.00	71	1898	12.02	71	1930	12.12	72	1895	12.92	77	1922	13.37	79	11	6	4	1
Wash. west.	56.98	1922	43.02	76	1890	43.17	76	1929	44.01	77	1905	46.18	81	1911	46.81	82	1923	47.25	83	9	3	0	0

*Eastern and western Oregon and Washington means are for period of record, 1890-1934.
(From unpublished Drought Report by the U. S. Geological Survey).

ficiency from lowest to highest. In a few states there were fewer than six years in which precipitation was as low as 85 per cent.

Since 1880 there have been 38 drought years in the humid states and 29 in the semi-arid states—that is, years during which the precipitation in one or more states was 85 per cent or less of the mean, as is indicated on the charts. Table II lists the humid and semi-arid states for fourteen years, when large areas had deficiencies in precipitation of 15 per cent or more, with the per cent of area in which the deficiency existed arranged both according to magnitude of area affected and chronologically.

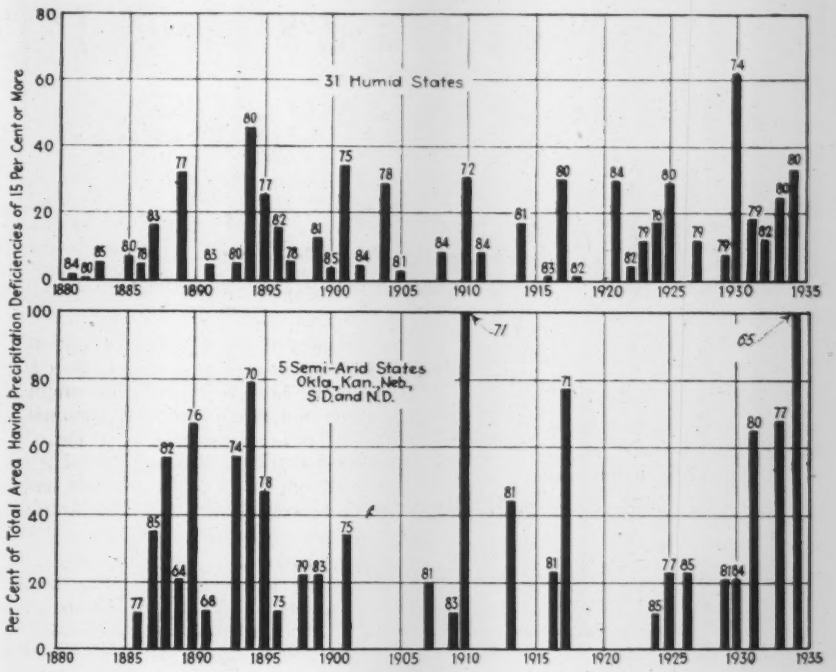
TABLE II.—YEARS WHEN LARGE AREAS OF THE HUMID AND SEMI-ARID STATES HAD A DEFICIENCY IN PRECIPITATION OF 15 PER CENT OR MORE

In Order of Extent				Chronological Order		
Number	Year	Area Affected Per Cent	Precipitation in Area of Deficiency in Per Cent of Mean	Year	Interval Between Deficiency, Years	Order of Extent of Area Affected
Humid States						
1	1930	62.6	74	1889	..	5
2	1894	45.4	80	1894	5	2
3	1901	34.3	75	1895	1	11
4	1934	32.4	80	1896	1	13
5	1889	32.2	77	1901	5	3
6	1910	30.5	72	1904	3	10
7	1917	30.3	80	1910	6	6
8	1921	29.7	84	1917	7	7
9	1925	29.6	80	1921	4	8
10	1904	28.9	78	1925	4	9
11	1895	27.0	77	1930	5	1
12	1933	23.8	80	1931	1	14
13	1896	17.7	82	1933	2	12
14	1931	17.5	79	1934	1	4
Semi-Arid States*						
1	1934	100	65	1887	..	12
2	1910	100	71	1888	1	8
3	1894	79.6	70	1890	2	5
4	1917	77.7	71	1893	3	9
5	1890	68.4	76	1894	1	3
6	1931	66.5	77	1895	1	10
7	1933	65.1	80	1901	6	13
8	1888	57.2	82	1910	9	2
9	1893	57.2	74	1913	3	11
10	1895	44.7	78	1914	1	14
11	1913	44.1	84	1917	3	4
12	1887	34.9	85	1931	14	6
13	1901	34.9	75	1933	2	7
14	1914	34.9	83	1934	1	1

* North Dakota, South Dakota, Nebraska, Kansas, and Oklahoma.

In the 31 humid states the time interval betwen drought years has ranged from three to seven years. During seven years the drought areas ranged from 30.3 to 62.6 per cent of the total. When judged by the extent of the area affected and the deficiency of precipitation, 1930 was the most severe drought year of record. The next two severe drought years were 1894 and 1901.

In five semi-arid states, North Dakota, South Dakota, Nebraska, Kansas and Oklahoma, the time interval between droughts has ranged from two to eight years. During seven years the drought area ranged from 65.1 to 100 per cent of the total, and there were two years in which it was 100 per cent. During seven other years the drought area ranged from 34.9 to 57.2 per cent of the total. For these states, from the standpoint of the extent of the area affected and the deficiency of precipitation, 1934 is the most severe drought year of record. The next two severe drought years were 1894 and 1910. There was very little difference, however, between conditions in 1934 and 1910, as in both years 100 per cent of the area was affected and the mean precipitation for the drought area in 1934 was 65.2 per cent of normal, and in 1910 it was 70.6 per cent of normal. In general, the areas affected differ from year to year.



DROUGHT AREAS, measured as per cent of total area, affected by droughts since 1881. Small figures indicate the percentage of normal rainfall in the area affected.

The precipitation in the semi-arid sections of the country is so small that vegetation is affected in a large measure not only by fluctuations in the total annual precipitation but also by unsatisfactory distribution throughout the year. These areas, which may be for alternate years either humid or arid depending upon water conditions, were originally natural range and produced, in the main, only vegetation for stock raising, which was successfully carried on for many years. In 1873 the Director of the U. S. Geological Survey, in testifying before a congressional committee on public lands, indicated that most of the country lying west of Minnesota was an arid region and in the main could not be used for agriculture except by irrigation. While irrigation has been successfully carried on in the arid sections, unfortunately it has not been found feasible in most semi-arid sections. Five years later he reiterated this statement before another committee. The U. S. Weather Bureau has repeatedly expressed a caution against any greater extension of agriculture in the semi-arid states.

Bunching of drought years

The reason for the failure to heed these warnings is indicated on the accompanying diagram, which shows that since 1880 drought years in the semi-arid states have been more or less bunched, with several years of no drought between each group. The periods in which no droughts occurred were 1880 to 1885, 1902 to 1906, and 1918 to 1923. In non-drought years these semi-arid areas were especially adapted to the growing of wheat, and during the good years large areas were converted from range to wheat land. It is this section that suffered so greatly during the present drought period. The drought period 1930-1934 again calls attention to the ever-present danger to many types of agriculture that have been undertaken in the semi-arid regions and that have not fully succeeded. If we are to judge the future by the past, there is no indication that they can be continued with any assurance of success under the present methods of farming. The success of agriculture in the semi-arid as in the arid sections depends largely upon the character of the development.

Additional Data on Model Tests for Boulder Dam Spillways

Discussion of tests made on models by Swiss laboratory defends design used for project—Full discharge will not place tunnel under pressure

By D. C. McConaughy
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STATEMENTS made and designs shown in the article by Drs. E. Meyer-Peter and Henry Favre on hydraulic-model tests for Boulder Dam spillways (*ENR*, Oct. 25, 1934, p. 520) may perhaps leave some doubt in the minds of engineers as to the adequacy of the design actually used. The authors are incorrect in stating that, "The lower part of the tunnels must act as pressure tunnels." Their model fails to meet a basic requirement, satisfaction of which was the main reason for the use of the models. The writer therefore proposes to discuss the shortcomings of the authors' design and their remedies.

The purpose of the model tests made by the Bureau of Reclamation was only incidentally to verify the position of the mean water surface, tests already having shown the reliability of existing formulas. The real purpose was to insure flow smooth enough to prevent choking of shaft and tunnel by waves and surges. This proved a difficult problem; for it, the authors' model provides no solution. The formula presented is said to include two factors neglected in that proposed by Julian Hinds in 1926 (*Trans.*, Am. Soc. C.E., Vol. 89); these are friction and the velocity component, parallel to the channel, of the entering water. A reading of Mr. Hinds' paper will show that he does include friction, in a way practically identical with that used by the authors. In the present case the water enters the channel so nearly at right angles to its axis that the velocity component is negligible.

The precise mean water surface, while of great value, is not the only information required for the design of this type of structure. The extremes are also very important in a practical design, and no formula for computing these has as yet been developed. Nor will the structure of minimum dimensions, ample in actual capacity though it may be, necessarily give satisfactory hydraulic conditions. The dimensions of a channel cannot, therefore, be determined exclusively by any formula but must depend largely on the judgment of the designer; and only in the most experienced hands will the analytical treatment, as stated by the authors, lead more directly to the desired solution than the test by models.

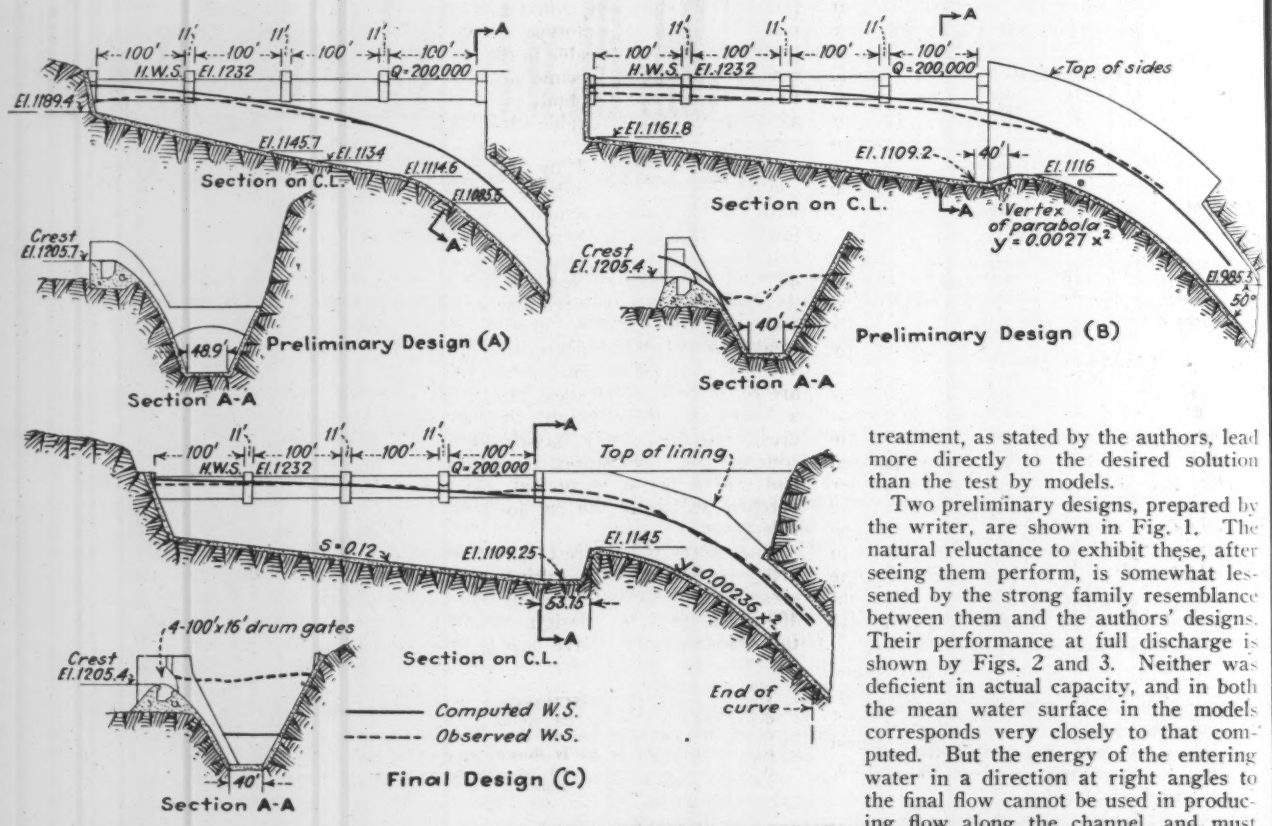


FIG. 1—EVOLUTION of spillway design, showing the longitudinal sections of the original models (top) and the final design as constructed (bottom).

Two preliminary designs, prepared by the writer, are shown in Fig. 1. The natural reluctance to exhibit these, after seeing them perform, is somewhat lessened by the strong family resemblance between them and the authors' designs. Their performance at full discharge is shown by Figs. 2 and 3. Neither was deficient in actual capacity, and in both the mean water surface in the models corresponds very closely to that computed. But the energy of the entering water in a direction at right angles to the final flow cannot be used in producing flow along the channel, and must be dissipated before smooth flow can be established. The channel must therefore act as a stilling pool as well as an escape channel, and in this respect both designs were shown by model tests to be faulty.

Nearly 100 different experiments were made with vanes, baffles and similar devices in an effort to secure the necessary smoothness of flow in the shaft; and while several met with success, the best results, all things considered, were obtained by the devices used in the final design—namely, the cross-weir or step across the channel

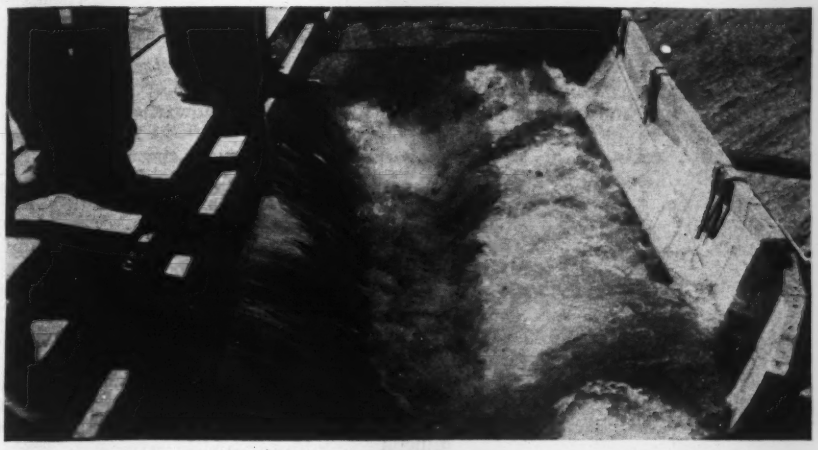


FIG. 2—CAPACITY DISCHARGE (200,000 sec.-ft.) in the preliminary model indicated in Fig. 1 (top left).

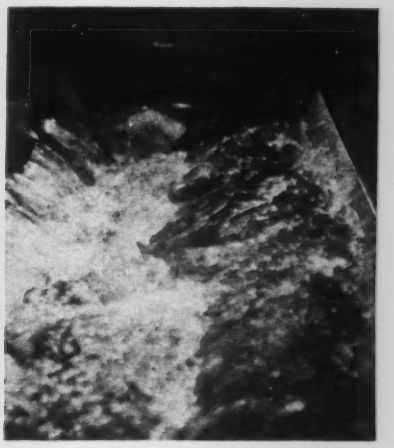


FIG. 3—DISCHARGE of 200,000 sec.-ft. in the modified model design shown in Fig. 1 (top, right).

and the shoulder on the crest side at the downstream end of the weir. The flow in a model of the final design at full discharge is shown in Fig. 4, and the effect of the two features may be seen by contrast with Fig. 3. The water surface could have been smoothed even more by means of baffles, but these would have added considerably to the cost. As the condition without them was quite good, they were omitted.

The cross-weir results in no higher water-surface elevation at the upper end of the channel than would exist if it were removed. The drop in water surface along the channel is determined by the longitudinal velocities; and the enlargement in wetted area due to the increase in depth caused by the cross-weir reduces these velocities so that the drop in water surface is decreased enough to compensate for the higher water surface at the lower end. The shoulder takes the place of the nappe and prevents a sudden enlargement in effective cross-section at the end of the weir, which was found to cause an objectionable wave. These features also result in considerable economy.

Smooth flow in the shaft and tunnel is necessary. The rough flow produced by the designs shown in Fig. 1 (*A* and *B*) resulted, in the largest models (1:20 scale), in noticeable vibration, which no doubt would be greatly magnified in the prototype. This vibration was not noticeable in the smaller models, although the same flow conditions existed. Models built to too small a scale are undesirable in several respects.

The gates proposed by the authors are about 26.5 ft. high as compared with the 16-ft. gates actually used. The lower gates have several advantages. For floods that are estimated to occur about 1 per cent or less of the time, their action is not required and the spillway becomes a free crest. Being smaller, they are cheaper in themselves; and since, at maximum discharge, they are retracted into the crest, they require a lower minimum height of concrete crest, resulting in a saving in both concrete and excavation, as well as in less water pressure against the weir perched on top of the cut forming the side channel.

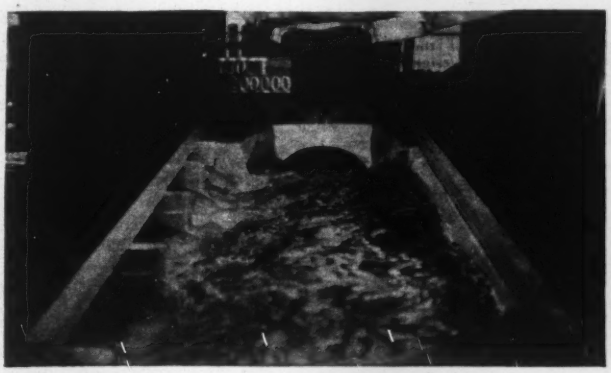
The shape of the crest shown in the article is inefficient, resulting in a net crest length of 456 ft. as compared with 400 ft. in the actual design. The additional length would have been very expensive, not only on account of the enormous size of the structures, but also because they were set between projecting points of high ground on the canyon walls, so that the increase in excavation would have been disproportionate to the increase in length.

Nor does the writer agree that improvement in the shape of piers shown by the authors is not worth while. With long gates, the influence of the piers on the discharge is naturally small; but any improvement that can be so easily made and that results either in a shorter structure or a lower required head, even if small, is worth while.

The most noteworthy feature of the article under discussion, however, is the statement on p. 521: "It is evident that, with the given dimensions and conditions of gradient, the lower part of this tunnel must act as a pressure tunnel."

This is to say that the flow changes suddenly from partial depth to flow under pressure. Such a condition, as the authors point out, would be accomplished by great losses of energy and probably would be considered very bad by most designers. Provided the tunnels are of sufficient area, whether or not this would occur depends on the height of tailwater in the river below the outlet portals. The maximum depth of flow in the tunnels will occur at the

FIG. 4—CAPACITY DISCHARGE in final design model, showing the spilling effect of the modified design in the channel. The test on the 1:20 model is shown on the left, and the capacity test on the 1:60 model at the right.



outlet portals where it is equal to about three-fourths the tunnel diameter.

The depth of tailwater would need to be increased about 30 per cent before sealing of the tunnels would occur, even under present conditions. A characteristic of the Colorado River is the large amount of scour that always takes place at flood stages, even with the large amount of silt present in the water of the uncontrolled stream. If allowance is made for the additional permanent scouring effect of large volumes of practically clear water on the fine sand and silt of the streambed, the increase in depth would have to be even greater.

Models of these structures were built to four different scales—namely, 1:150, 1:100, 1:60 and 1:20. In none of these was there any indication that such a condition as predicted by the authors would exist. The explanation of the difference in results must lie in a difference in the models. What this difference might be can only be surmised; it probably has to do with tailwater conditions.

The determination of the amount of entrained air, mentioned by the authors, was one of the most vexing problems involved and one for which no thoroughly satisfactory solution was reached. Available data are mostly based on indirect measurements, which are hard to make with any degree of precision on account of the high velocities involved and the turbulence usually present; and they are therefore of very doubtful value. It seems self-evident, however, that the amount of air entrained would depend largely on the amount of surface turbulence and the distance that the water must fall between the reservoir water surface and that in the channel, these being related quantities. The authors' design shows a drop of about 60 ft. at the lower end of the weir crest; in the actual design this distance is about 22 ft. Considerable work was done in investigating air entrainment, and indications were very strong that the per cent of air actually entrained at maximum discharge would be much below that allowed for.

The writer is inclined to agree with the conclusions reached by the authors, as given in the closing paragraph of the article, that a larger model would have been desirable and that their experiments "cannot claim to have solved the problem in every particular."

Shrinkage of Hydraulic Fill in Miami Conservancy Dams

Recapitulation of shrinkage records obtained since 1921 on Huffman and Germantown flood-protection dams of the Miami Conservancy District

By C. H. Eiffert
Chief Engineer, Miami Conservancy District, Dayton, Ohio

SHRINKAGE in the Huffman Dam of the Miami Conservancy District was reported in *Engineering News-Record*, Jan. 27, 1935, p. 145. (The figures arranged as in Table I, carried the record to and including the reading of Sept. 30, 1926. Table I includes the last reading of the former table and the readings since to and including that of Feb. 1, 1935.—Editor.) Readings on the benchmarks have been continued since that time, and, as some further settlement has occurred, the figures may be of interest. In addition, data on the Germantown Dam, which were not previously reported, are included here.

TABLE I—RECORDS OF ELEVATIONS AND SETTLEMENT AT HUFFMAN DAM
(Continued from previously published tabulation in ENR, Jan. 27, 1927)

Pipe No.	Location	Sept. 30, 1926			May 1, 1928	Dec. 9, 1930	May 25, 1931	Nov. 2, 1931	Nov. 25, 1932	Sept. 10, 1934	Feb. 1, 1935		
		Elev.	Total Settle	%	Elev.	Elev.	Elev.	Elev.	Elev.	Elev.	Elev.	Total Settle	%
1	West	809.90	0.10	0.40	809.89	809.86	809.84	809.85	809.85	809.84	809.839	.16	0.64
8	"	809.84	0.14	0.56	809.82	809.82	809.79	809.78	809.78	809.77	809.783	.22	0.88
15	"	809.92	0.10	0.40	809.89	809.89	809.87	809.85†	809.85	809.84	809.861	.16	0.64
7	East	809.83	0.17	0.68	809.82	809.79	809.77	809.77	809.78	809.77	809.762	.24	0.96
14	"	809.90	0.12	0.48	809.90	809.88	809.85	809.84	809.85	809.84	809.847	.17	0.68
22	"	809.87	0.16	0.64	809.85	809.83	809.79†	809.79	809.78	809.79	809.784	.25	1.00
2	West	819.89	0.11	0.31	819.89	819.85	819.84	819.84	819.84	819.83	819.837	.17	0.49
9	"	819.85	0.15	0.43	819.84	819.83	819.80	819.80	819.80	819.79	819.798	.20	0.57
16	"	819.88	0.12	0.34	819.85	819.85	819.83	819.82	819.82	819.80	819.825	.18	0.51
6	East	819.82	0.18	0.51	819.81	819.78	819.76†	819.77	819.77	819.76	819.758	.24	0.69
13	"	819.84	0.16	0.46	819.84	819.82	819.79	819.79	819.79	819.78	819.787	.21	0.60
21	"	819.81	0.19	0.54	819.80	819.79	819.76	819.75	819.75	819.75	819.749	.25	0.71
3	West	829.75	0.25	0.56	829.75	829.70	829.69†	829.69	829.69	829.68	829.680	.32	0.71
10	"	829.71	0.29	0.64	829.69	829.66	829.65	829.65	829.65	829.63	829.638	.36	0.80
17	"	829.76	0.24	0.53	829.73	829.71	829.69	829.68	829.67	829.65	829.664	.34	0.76
5	East	829.73	0.27	0.60	829.72	829.68	829.66†	829.67	829.67	829.65	829.651	.35	0.78
12	"	829.72	0.28	0.62	829.73	829.69	829.67	829.66†	829.67	829.65	829.659	.34	0.76
20	"	829.80	0.20	0.44	829.78	829.75	829.73†	829.73	829.72	829.72	829.722	.28	0.62
4	West	839.59	0.41	0.75	839.58	839.52	839.51†	839.51	839.51	839.49	839.485	.51	0.93
19	East	839.87	0.13	0.24	839.85	839.81	839.79	839.79	839.78	839.77	839.770	.23	0.42
18	West	849.68	0.32	0.49	849.65	849.59	849.57†	849.57	849.56	849.53	849.537	.46	0.71
11	East	849.61	0.39	0.60	849.59	849.51	849.51	849.50	849.50	849.47	849.475	.53	0.81
Average				0.51									0.71

Shrinkage progress

The locations of benchmarks at the two dams are shown by Fig. 1. On the Huffman Dam these benchmarks consist of iron pipes 7 ft. long driven in the fill at various elevations during the progress of the work. At Germantown only four benchmarks were installed, and these were in the top of the dam after its completion. They originally consisted of 2x4-in. stakes, the tops being about 1 ft. below the road surface. When levels were taken on them in 1929, it was seen that they were in rather poor condition and would not last much longer. Iron pipes 4 to 6 ft. long were driven about 1 ft. distant from the old stakes and to the same elevations. Since that time the elevations have been taken on the new stakes.

The progressive shrinkage or settlement of these dams may best be seen from the diagram (Fig. 3). The points platted represent the average settlement in per cent of all benchmarks, at given intervals of time. These curves show a somewhat greater settlement or shrinkage at Huffman and a considerable difference at the beginning of the period. The writer believes that these differences may both be largely due to the fact that at Germantown no bench-

TABLE II—RECORDS OF ELEVATIONS AND SETTLEMENT AT GERMANTOWN DAM

Pipe No.	H	Nov. 15, 1920 Elev.	Apr. 28, 1921 Elev.	Aug. 3, 1922 Elev.	Nov. 21, 1929 Elev.	Dec. 1, 1930 Elev.	May 23, 1931 Elev.	Oct. 31, 1931 Elev.	Nov. 26, 1932 Elev.	Sept. 11, 1934 Elev.	Feb. 2, 1935 Elev.	Feb. 2, 1935 Total Settle	Feb. 2, 1935 P.C.
1	32	828.41	828.33	828.31	828.19	828.21	828.20	828.20	828.19	828.20	828.20	0.21	0.66
2	101	828.13	827.88	827.77	827.50	827.51	827.50	827.49	827.48	827.47	827.47	0.66	0.65
3	98	829.28	829.05	828.93	828.77	828.78	828.77	828.76	828.75	828.74	828.73	0.55	0.56
4	56	828.48	828.36	828.30	828.15	828.15	828.15	828.14	828.14	828.14	828.14	0.34	0.61
Average													0.62

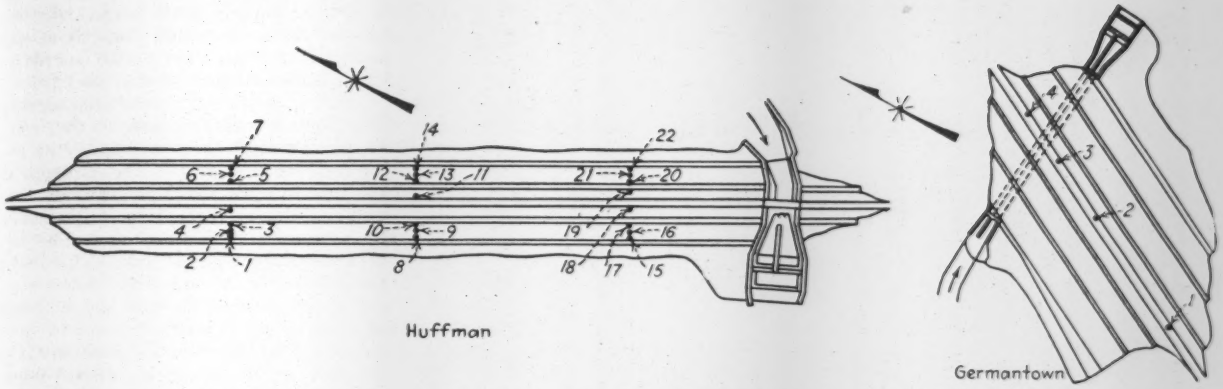


FIG. 1—BENCHMARK locations for measuring shrinkage on Huffman and Germantown dams of the Miami Conservancy District.

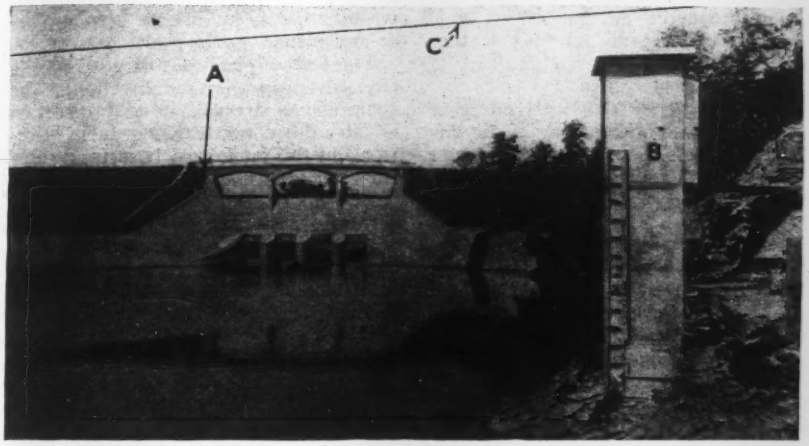


FIG. 2—TAYLORSVILLE DAM showing (A) location of surface connections to pressure cells, (B) automatic stage recorder, and (C) cableway for stream measurements.

marks were placed until after the completion of the entire dam, while at Huffman they were placed during the progress of the work. For this reason probably some of the early and more rapid settlement does not show on the Germantown record.

Earth-pressure records

During the construction of the Miami Conservancy District dams, Goldbeck pressure cells were installed to determine vertical and horizontal pressures of core material. A report on these was made by C. H. Paul (*Trans.*, Am.Soc.C.E., Vol. 85, p. 1181). Supplemental information concerning them was given in *Engineering News-Record*, Dec. 18, 1930, p. 954.

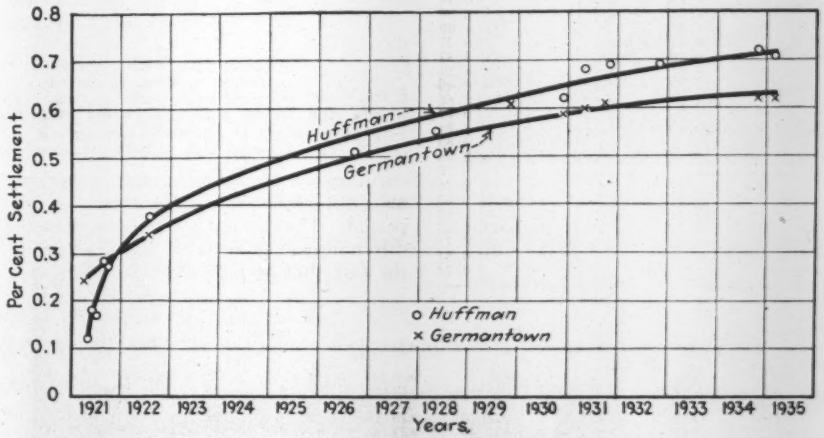


FIG. 3—PROGRESSIVE SHRINKAGE of Huffman and Germantown dams in terms of average settlement of all benchmarks.

Due to methods of installation and other causes, all of the cells except those at the Taylorsville Dam became inoperative near the close of the construction period. At Taylorsville they were placed adjacent to the concrete wall of the outlet works at the location indicated in Fig. 2, and the operating pipes were brought carefully to the surface. The cells were tested in 1922, again in 1925, and again in September, 1934. On the last-mentioned date all but one of the cells were still in working order. The pressures recorded at that time were practically identical with those taken in 1925, and the record remains about as shown in *Engineering News-Record*, Dec. 18, 1930, Fig. 1, p. 955. Analyses of the core materials of these dams are given in the above-mentionel articles.

Unified State System of Roads Proposed in California

Legislation which Governor Frank F. Merriam of California has recommended for passage at the present session of the legislature in that state would (1) declare all county roads to be state highways and would provide for expenditure by the state of the highway funds now allocated to the counties; (2) double the present allotment to cities from the gasoline tax fund, thus raising the amount guaranteed to the cities from ¼ to ½ cents per gallon; and (3) prohibit the issuance of county bonds for highway work so that after retirement of the counties' present indebtedness for road bonds no further taxes would be collected by counties.

Active opposition to this program has been taken up by several influential organizations. A compromise bill may be proposed by the Auto Club of Southern California.

Provisions of the plan proposed by the governor in several respects are similar to those of the plan recently developed in the state of Washington and described in *ENR*, Jan. 3, 1935, p. 25.

Designing Composite Steel and Concrete Bridge Slabs

Analysis of test data for light-weight slabs of the concrete-filled steel-grid type, such as were used on two recent bascule spans in New Jersey, develops a rational design procedure and fixes satisfactory working stresses

By Shortridge Hardesty
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TWO BASCULE SPANS over the Hackensack River in the villages of Secaucus and Little Ferry, designed by the writer's firm for the state highway department of New Jersey and completed during the past year, utilize light-weight composite roadway slabs of the concrete-filled, steel-grid type. The particular slabs used are 2½ in. thick and consist of a ¼-in. bottom steel plate to which are welded (by intermittent fillet welds 2 in. long and 6¼ in. on centers on both sides of each bar) 2¼x⅜-in. vertical shear bars, 7⅛ in. on centers, between which are riveted four lines of 1x⅜-in. reticulated bars, the slab being filled with concrete struck off flush with the upper surface of the bars. All steel is copper-bearing.

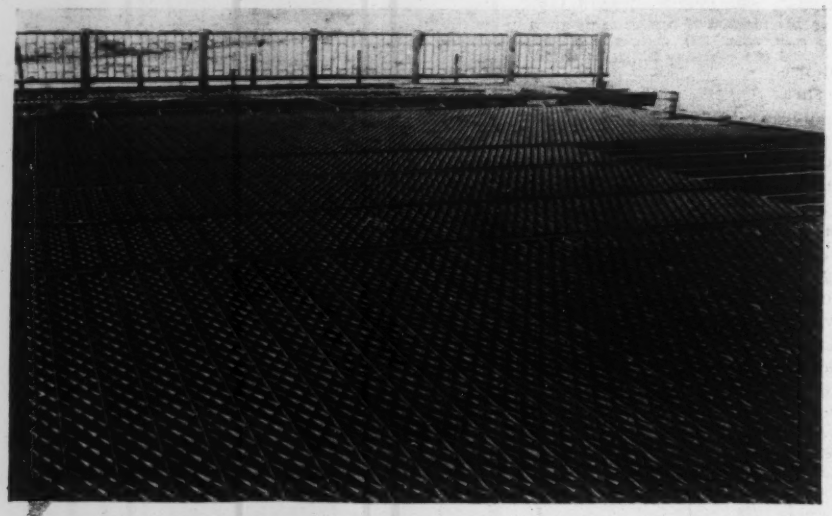
The completed slab weighs about 45 lb. per sq.ft.

For the type of slab used on these bridges, the size and spacing of the different elements can be varied to suit any given conditions. The over-all depth of the slab will usually be from 2¼ to 3½ in. The shear bars are the principal carrying members, and run at right angles to the supports. The strength of the slab is determined mainly by the compressive stresses in the concrete at the top of the slab, and by the welds connecting the shear bars to the bottom plate. Since the welds can be increased as desired, the concrete stress is the limiting factor. As the shear bars are efficient in resisting moment, thickening them is an effective way of increasing the strength of the slab. Such thickening also increases greatly the strength of the steel framework, which strength is valuable during construction and also later in the event of any damage to or weakness in the concrete at any spot. The stresses in the bottom plate are comparatively low and its thickness is largely governed by corrosion considerations. The reticulated bars are thick enough to resist undue distortion and are effective in carrying compressive stresses, although necessarily at somewhat reduced efficiency. The shear bars evidently carry a large part of the shear, which is important in the event of weak bond between the concrete and the bottom plate at any point.

The bottom plate affords good tensile strength in a transverse direction. The compressive strength in that direction is fair, the concrete being fully effective and the reticulated bars reasonably so.

The flooring is usually furnished in sections about 4 ft. wide. Adjoining sections are well connected, the bottom plates and the shear bars by frequent welds and the reticulated bars by bolts. The sections are fastened to the supporting steelwork by plug welds or bolts in holes in the bottom plates.

STEEL-GRID FLOOR ready for concrete fill on bascule span over Hackensack River at Secaucus, N. J. A similar roadway, but without the streetcar tracks, was used on the new bascule at Little Ferry, N. J.

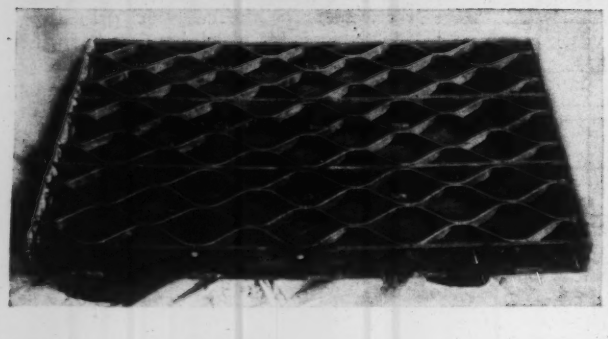


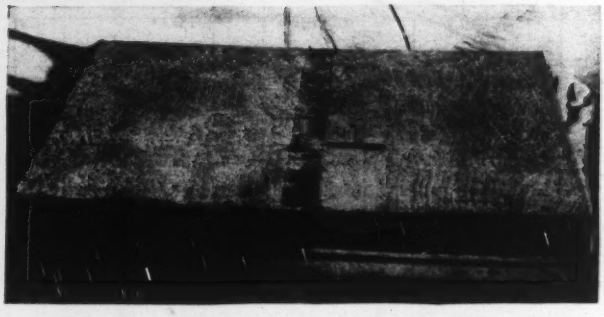
Test slabs

In a composite slab of this type it is necessary to determine by test the strength of a unit of the flooring and the distribution of the stress over the concrete and steel elements, as well as the transverse distribution of concentrated loads. The Kerlow Steel Flooring Co., manufacturer of the slabs, had several tests made at Columbia University in 1932, to secure data relative to the strength and stress distribution. The writer made some calculations to interpret the results of these tests, which may be of interest, as similar methods can be applied to other composite-floor construction.

In this investigation each test slab was 36 in. long, 21½ in. wide and 3⅛ in. thick, and consisted of a ⅛-in.

SLABS after testing, showing, at the left, the make-up of the steel elements, consisting of a bottom plate and vertical shear plates separated by reticulated bars and, at the right, a similar slab with the concrete fill in place.





bottom plate, four $2\frac{3}{4}$x$\frac{5}{16}$-in. shear bars, and twelve lines of 1x$\frac{3}{16}$-in. reticulated bars. Three slabs were tested, one being filled with 1:2:3 concrete after the steel members had been coated with paraffin, representing the condition of poor bond between the steel and the concrete; one being tested unfilled; and the third being filled with concrete with the steel clean and uncoated. Each slab contained 23.6 lb. of steel, and the filled slabs 31.4 lb. of concrete per sq.ft., or a total of 55 lb. per sq.ft. The slabs were tested on a 30-in. span, the load being applied in the center over the full $21\frac{1}{2}$-in. width. The center deflection was measured by an Ames dial; the tensile stress in the bottom plate and the compressive stress at the top edge of one of the inner shear bars were measured by Huggenberger extensometers; and the tensile stress in the lower edge of one of the outer shear bars, by a Berry strain gage.

The test results were as follows, in terms of the total load at span center:

Kind of Slab	Elastic Limit, Lb.	Ultimate Strength, Lb.
Unfilled slab	18,000	46,000
Filled slabs		
Paraffin-coated	20,000	103,000
Uncoated	35,000	100,000

The elastic limit was taken as the point at which the rate of deflection showed a marked increase, while the ultimate load represents the point at which the deflection increased rapidly with practically no increase in load. In the case of the filled slab with steel uncoated, the elastic limit represented the beginning of a marked flow in the welds connecting the shear bars to the bottom plate. The rate of deflection increased still more rapidly as various elements of the steel reached the elastic limit. The slab in which the steel was coated with paraffin showed practically the same ultimate strength as the slab with the steel uncoated, but the elastic limit was lower, indicating some movement of the concrete with respect to the steel because of the weakened bond.

Examination of the various slabs after testing disclosed no failures in welds or riveted connections.

Analysis of test results

The slab filled with concrete, with steel uncoated, represents actual service conditions, and the results pertaining to it were analyzed by the writer as given by the calculations in Tables 1 to 5.

Table 1 gives the test results, together with the moment of inertia of the slab as computed from the deflections, and the maximum steel stresses in the bottom of the bottom plate and in the top and bottom of each shear bar. Only 0.8 of the observed deflection has been used in the calculations, to allow for shear deflection and movement at the supports. The theoretical value of the moment of inertia is 16.8 (as developed in Table 2). It will be noted in Table 1 that the observed value, based on the successive deflections, stands at an average of 15.0 up to a load of 30,000 lb., breaks to 8.0 at 40,000, and then remains constant up to a load of 60,000, beyond which point it drops rapidly. It will also be noted that between 35,000 and 40,000 lb. there was a sudden increase in the tension in the bottom edge of the outer shear bar, due to flow in the welds connecting it to the bottom plate; the elastic limit was passed in the upper edges of all shear bars at about the same time. The elastic limit in the bottom plate was passed between 70,000 and 75,000 lb.

Table 2 gives the moment of inertia and section moduli for a $7\frac{1}{16}$-in. width, a 12-in. width, and the full $21\frac{1}{2}$-in. width of the test slab. The moment of inertia is computed on the assumptions that $n = 12$ and that the reticulated bars are 70 per cent effective.

Table 3 shows the probable unit stresses in the various elements of the test slab under a load of 25,000 lb., as derived from the test measurements; and also the unit stresses under the same load, as determined from the computed section moduli. It will be noted that the unit stresses as computed on the two bases are in reasonably close agreement, thus checking the assumptions regarding n and the effectiveness of the reticulated bars.

In the computations for the probable unit stresses in the test slab, the unit stresses in the shear bars and the bottom plate were known from the test data, as was also the applied moment. The unit stress in the concrete and the efficiency of the reticulated bars were

TABLE 1—ANALYSIS OF TEST ON CONCRETE-FILLED SLAB

Loads		Deflection D		I from D		Bottom Plate		Inner Vertical Bars			Outer Vertical Bars		
								Strain Δl	Extr. Fiber Stresses		Strain Δl	Extr. Fiber Stresses	
Total	Total Less Initial	Total	Diff.	Total	Diff.	Strain Δl	Stress $E\Delta l$		Top $= 1.2^* E\Delta l$	Bott.		Top	Bott. $= 1.2^* E\Delta l$
1,000	0	.000	.000			.00000	0	.00000	0	0	.00000	0	0
5,000	4,000	.005	.005	17.8	17.8	.00004	1,200	.00007	2,500	2,000	.00003	3,400	1,100
10,000	9,000	.013	.008	15.4		.00011	3,300	.00017	6,100	5,700	.00007	9,300	2,500
15,000	14,000	.020	.007	15.6	15.0	.00019	5,700	.00028	10,100	8,100	.00013	13,500	4,700
20,000	19,000	.028	.008	15.1	av.	.00026	7,800	.00039	14,000	11,500	.00015	20,100	5,400
25,000	24,000	.035	.007	15.2		.00035	10,500	.00050	18,000	13,800	.00023	23,500	8,300
30,000	29,000	.042	.007	15.3		.00045	13,500	.00062	22,300	15,800	.00034	25,900	12,200
35,000	34,000	.051	.009	14.8	12.3	.00056	16,800	.00077	27,700	18,700	.00044	30,600	15,800
40,000	39,000	.062	.011	14.0		.00065	19,500	.00092	33,200	23,200	.00083		29,800
45,000	44,000	.075	.013	13.1	8.0	.00069	20,700	.00110	39,600	28,700	.00124		44,300
50,000	49,000	.089	.014	12.2	av.	.00077	23,100	.00131			.00157		
55,000	54,000	.103	.014	11.7		.00086	25,800	.00157			.00198		
60,000	59,000	.118	.015	11.1		.00096	28,800	.00180			.00242		
65,000	64,000	.137	.019	10.4	6.0	.00109	32,700	.00207			.00300		
70,000	69,000	.156	.019	9.8	av.	.00116	34,800	.00231			.00363		
75,000	74,000	.179	.023	9.2	5.0	.00133	39,900	.00253			.00433		
80,000	79,000	.211	.032	8.3	3.4	.00168		.00273			.00503		
85,000	84,000	.245	.034	7.6	av.	.00196		.00283			.00562		
90,000	89,000	.289	.044	6.9	2.5	.00227		.00297			.00677		
95,000	94,000	.334	.045	6.3	av.	.00248		.00319			.00782		
100,000	99,000	.414	.080	5.3	1.4	.00382		.00382			.01002		

NOTES

Deflection from moment, D_M, assumed to be 0.8 of measured deflection D.

$$I = \frac{P l^3}{48 E D_M} = \frac{29.5^3 P}{48 \cdot 30{,}000{,}000 D_M} = \frac{P}{56{,}200 D_M} = \frac{P}{45{,}000 D}$$

Difference in fiber stresses in top and bottom edges of vertical bars of depth d

$$f = \frac{12 E d D_M}{l^2} = \frac{12 \cdot 30{,}000{,}000 d D_M}{29.5^2} = 413{,}000 d D_M = 330{,}000 d D = 330{,}000 \times 2.75 D = 910{,}000 D$$

*Factor 1.2 used because extensometers were not at points of maximum stress.

TABLE 2—THEORETICAL PROPERTIES OF FILLED SLAB, ASSUMING $n = 12$, AND RETICULATED BARS 70 PER CENT EFFECTIVE

$7\frac{1}{16}$-in. width		Areas A		$A_c + 12A_s$				Mom. of Inertia $I_c + 12 I_s$		Section Moduli	
Bott. pl.	$7.06 \times \frac{5}{16}$	2.21 sq.in.	$\times 12$	26.5 sq.in.	$\times$	0.156	4.2	0.986^2	25.8	Top	
									0.2	Concrete $63.8 \div 1.92 = 33.2$	
Vert. bars	$2.75 \times \frac{5}{16}$	0.86	$\times 12$	10.3	$\times$	1.375	14.2	0.545^2	3.0	Steel $33.2 \div 12 = 2.77$	
									6.5		
Reticul. bars	$4 \times 1 \times \frac{3}{16} \times 1.2$	0.90	$\times \frac{0.7}{1.2} \times 11$	5.8	$\times$	2.26	13.0	1.42^2	11.7	Bottom	
									0.5	Steel $6.38 \div 1.14 \times 12 = 4.67$	
Concrete	6.75×1.92	13.00	$\times 1$	13.0	$\times$	1.79	23.2	0.96^2	12.0		
									4.1		
Total				55.6	$\times$	0.83	46.2		63.8		
12-in. width—Multiply by 12/7.06				94.5 sq.in.	$\times$	0.83	78.5		108.4	Top Concrete = 56.5 steel = 4.71 Bott. Steel = 7.94	
$21\frac{1}{2}$-in. width = 3 units @ $7\frac{1}{16}$ in. + 4th vert. bar $2\frac{1}{4}$ in. x $\frac{5}{16}$ in.		178.3 sq.in.	$\times$	0.86	= 152.6				201.8	Top Concrete = 106.8 steel = 8.90 Bott. Plate = 14.37 Bott. Vert. bar = 19.5	

$I_c = 201.8 \div 12 = 16.8$

TABLE 3—UNIT STRESSES IN TEST SLAB, FOR LOAD OF 25,000 LB.

Values from Test Measurements

Moment = 25,000x29/4 = 181,000 in.-lb.
D = 0.037 in. I_s = 15.0

	T and C	Moment
Bott. pl. 6.7x12.300×0.90/1.06	70,000x1.22	85,500 in.-lb.
Inner vert. 13,800×0.62x0.56	4,800x0.68	3,300
Outer vert. 11,100×0.62x0.46	3,200x0.76	2,400
Total tension	78,000	
Inner Vert. 19,800×0.62x0.81	10,000x1.15	11,500
Outer vert. 22,500×0.62x0.92	12,800x1.07	13,700
Reticul. 0.7x2.25×20 700x1.19/1.69	23 000x1.24	28 500
Concrete 20.25x1 870×0.85	32 200x1.13	36 400
Total compression	78 000	181 300

Values Computed from Properties of Section

Moment = 181,000 in.-lb. D = 0.033 in I_s = 16.8
Concrete f_c = 181,000 ÷ 106.8 = 1,700C
Top of vert. bars f_s = 181,000 ÷ 8.9 = 20,400C
Bott. edge of vert. bars f_s = 181,000 ÷ 19.5 = 9,300T
Bott. plate f_s = 181,000 ÷ 14.37 = 12,600T

welds, it was decided that they should be made 2 in. long, and this length was used in the slabs for the Hackensack River bascule spans.

While the foregoing data indicate that n should be taken as 12, it has been customary in the design of such composite slabs of steel and concrete to consider that n=15, in line with the usual assumption for ordinary reinforced-concrete beams. Using 15 rather than 12 results in a considerable increase in the value of the moment of inertia and of the section modulus for concrete stresses, and therefore a reduction in the computed value of the concrete stress. For n=15 the computed compressive stress in the concrete in the test slab, for a moment of 68,000 in.-lb. per foot width, would be about 1,000 lb. per sq.in. If, therefore, design is to be based on the assumption that n=15, the compressive stress in the concrete should not be permitted to exceed 1,000 lb. per sq.in.

For the 2½-in. slabs used on the Hackensack River bascules, the computed properties per foot of width are:

		n = 12	n = 15
Moment of inertia I_c		59.4	70.7
$I_s = (I_c \div n)$		4.95	4.71
Section moduli			
Top fibers	Conc.	38.5	44.7
	Steel	3.21	2.98
Bottom fiber	Steel	5.15	5.12

Moment of resistance (f_c 1,200 and n 12) is 38.5 × 1,200 = 46,200 in.-lb.

Fiber stresses (M = 46,200 in.-lb.)

		n = 12	n = 15
Top fibers	Conc.	1,200	1,030
	Steel	14,400	15,450
Bottom fiber	Steel	8,980	9,030

In the Hackensack bascule floors the maximum slab span, center to center of the stringers, is 2 ft. 8 in. For a 16,000-lb. wheel with 30 per cent impact, assumed applied to the surface of the slab over the area of a 15-in. circle, the moments by the Westergaard formulas and the concrete stresses are:

	Ends Free	Ends Fixed	One End Fixed One End Free
Moments, In.-Lb. Per Foot Width	41,400	24,000	32,700
Stresses			
n = 12	1,075	625	850
n = 15	925	537	732

Since the slab is continuous over one or both supports and is welded to the stringers, it may be considered as having one end fixed and one end free. The concrete stress, for n=15, is therefore found to be something less than 750 lb. per sq.in.

Conclusions

From the analysis of test data given in this article, it would appear that Kerlow type M slabs can be designed by computing the theoretical properties of the section, and allowing f_c=1,200 if n is assumed to be 12, or 1,000 if n is taken as 15.

The United States Bureau of Public Roads recommends f_c=750 for n=15, and permits the slab to be considered monolithic with the supporting stringers of the floor framing if effectively welded thereto.

The life and service of slabs of this type can be determined only by observing their performance in actual installations. The slabs on the Hackensack spans have stood up satisfactorily for the several months that they have been in use, and have always exhibited good riding qualities.

TABLE 4—THEORETICAL PROPERTIES OF UNFILLED SLAB, ASSUMING RETICULATED BARS 30 PER CENT EFFECTIVE

Width, In.	A	A (effective)	I_s	Section Moduli			
7¼	3.97 sq.in.	3.29 sq.in. × 0.41 = 1.33	2.83	Top	1.21	Bott.	3.93
12	.....	5.59 × 0.41 = 2.26	4.81	Top	2.06	Bott.	6.68
21	.....	10.83 × 0.48 = 5.15	9.81	Top	4.32	Bott.	12.40

TABLE 5—STRENGTHS OF SLABS AS DETERMINED FROM TESTS, AND CALCULATED UNIT STRESSES

	Concrete-Filled Slab					Unfilled Slab			
			Unit Stresses					Unit Stresses	
	Load Lb.	Moment In.-Lb.	Concrete Top	Steel Top	Steel Bott.	Load Lb.	Moment In.-Lb.	Top	Bott
Test slab 21½ in. wide									
Ultimate strength	100,000	725,000	.....	.....	.....	46,000	333,000	.....	.....
Elastic limit	35,000	254,000	2,400	29,000	18,000	18,000	130,000	30,000	10,500
Safe working load	17,500	127,000	1,200	14,500	9,000	12,000	87,000	20,000	7 000
Values for 1-ft. width	56.5 ÷ 106.8 = 53% of values for 21½-in-slab.					2.06 ÷ 4.32 = 48% of values for 21½-in-slab.			
Ultimate strength	53,000	384,000	.....	.....	.....	22,000	160,000	.....	.....
Elastic Limit	18.500	135,000	2,400	29,000	18.000	8,600	62,000	30.000	9,300
Safe working load	9,250	68,000	1,200	14,500	9,000	5,700	41,000	20,000	6,200

then adjusted until the compression on the section equaled tension, and the computed moment equaled the applied value.

In Table 4 are given the properties of the steel framework only, the reticulated bars being assumed 30 per cent effective. Finally, Table 5 gives the ultimate strength, elastic limit and working strength of the concrete-filled slab and the unfilled slab, together with the respective unit stresses as computed from the properties of the sections.

The working strength has been taken as one-half the elastic limit for the concrete-filled slab, and as two-thirds thereof for the unfilled slab.

The calculations indicate for the concrete-filled slab a safe value for the moment of resistance of 68,000 in.-lb. per foot width, corresponding to a compression of 1,200 lb. per sq.in. in the concrete, a compression of 14,500 lb. per sq.in. in the top edge of the shear bars, and a tension of 9,000 lb. per sq.in. in the bottom plate. The yield point of the unfilled slab occurs at a moment of 62,000 in.-lb. per foot width. This means that, even though the concrete should be injured or weakened at any point, the elastic limit of the steel should not be exceeded, and there will be no danger of complete failure.

In the test slabs the fillet welds fastening the shear bars to the bottom plate were only 1 in. long. Since excessive movement developed in these

Washington Monument Located in National Geodetic Network

During the period when the Washington Monument, in Washington, D. C., was encased in the scaffolding erected to facilitate its cleaning and repair, the U. S. Coast & Geodetic Survey took observations from the tip to locate its geographic position exactly. The triangulation party ascended to the 500-ft. level of the monument in the construction elevator, then climbed the ladder an additional 55 ft. to the wind-swept top. The theodolite was set up and angles were measured to the observable triangulation station. When the records were submitted to the Division of Geodecy for final computation, the monument was officially located at 38 deg., 53 min., 21.681 sec. in latitude, and 77 deg. 2 min., 7.955 sec. in longitude.

As a result, the monument need not be referred to as merely being near 15th St. & Constitution Ave., N.W. Further, the historic structure is now definitely placed in the national network of triangulation stations.

Utah Supreme Court Decisions Confuse Underground Water Law

Two recent rulings on cases involving artesian wells leave water users in doubt as to application of correlative rights, doctrine of prior appropriation or "property of the soil"

By C. J. Ullrich
Consulting Engineer, Salt Lake City, Utah

TWO recent decisions of the Supreme Court of Utah, handed down in cases involving artesian wells, have tended to confuse the question of underground water rights in the state. The rulings are of importance in indicating a trend in the opinion of the court on the fundamentals of Utah underground water law. Further, Salt Lake City is vitally interested because it obtains a substantial part of its municipal supply from artesian basins. Because of the growing importance of legal decisions pertaining to underground water supplies, the situation in Utah is briefly reviewed.

The Utah water-rights law was first enacted in 1903. It accepted the doctrine of prior appropriation, and under this law no rights to any of the public waters of the state may be acquired except by application through the office of the state engineer in the legally prescribed manner. This 1903 statute, which is still in force, states "Rights to the use of the unappropriated public waters in this state may be acquired by appropriation in the manner hereinafter provided and not otherwise," and declares that "The water of all streams and other sources in this state, whether flowing above or under the ground in known or defined natural channels, is hereby declared to be the property of the public subject to all existing rights to the use thereof."

Three years prior to the passage of this law the state supreme court, in the case of the Willow Creek Irrigation Co. vs. Michaelson, defined percolating water as "water intermingling with the ground or flowing through it by filtration or percolation or by capillary attraction." The court said further that percolating water "is but a component part of the earth, and as such has no characteristics of ownership distinct from the land itself." By this phrasing the court indicated that percolating water was a part of the soil itself and belonged to the owner of the land. Since that decision it has always been considered that percolating water was not public water in the state of Utah.

More than two decades later, in 1921, the supreme court, in the case of Horne vs. Utah Oil Co., defined the doctrine of correlative rights as applying to an artesian basin and held that owners of land underlaid by an artesian basin were entitled to draw water from that basin in proportion to the area of land that each owned in the basin. Subsequently, in 1925, the supreme court, in the case of Deseret Livestock Co. vs. Hooppiania, stated that as a result of the 1903 statute no water rights could be established to any public waters in this state except in the manner outlined by statute, including the proper filings in the state engineer's office.

As a result of these prominent decisions there has grown up in the state of Utah an accepted, although unwritten, water code which has excepted artesian and percolating waters from the classification of public water and assumed that the former was governed by the doctrine of correlative rights, and the latter was to be considered the property of the soil. Since the Horne decision hundreds of people who have developed underground water have spent money acquiring land to give them a sufficient water right, believing they were making their water rights secure under the aforesaid decision. These people did not file on the water, as they considered it private and not public water.

Recent decisions

On Jan. 3, 1935, the supreme court in its decision in the Wrathall-Johnson case, ruled that the drilling of a well which resulted in stopping the artesian flow of a prior well was to be considered a cause for legal action, but added to the ruling additional explanations which tended to cloud what was assumed to be the definite position of artesian waters in the state law. In the case before the supreme court, which came up from the lower court on a sustained demurrer of the defendant that the complaint did not state sufficient cause for action, the facts showed that Wrathall had drilled a well on his own land and secured an artesian flow of water. Subsequently Johnson drilled a well on his property and developed an artesian flow. Not having sufficient water Johnson equipped his well with a pump to increase the natural flow. The operation of Johnson's well stopped the flow of Wrathall's well. The court was unanimous in its opinion that the complaint stated a cause for action.

Having indicated its decision in the case, the court continued to discuss the question in detail, which involved a consideration of the entire legal status of underground waters. There appeared to be disagreement between the judges on many of the points discussed, but the statements indicated the trend of the court on the subject. Analysing the text of the decision, two judges hold that the lower court should apply the doctrine of correlative rights in determining the rights of the litigants in this case. The opinion of two other judges may be construed as a reversal of the doctrine of correlative rights as laid down in the Horne case and a recognition of virtually all underground water as public water. These two judges go further and hold that no legal filings in the state engineer's office are necessary to appropriate the public waters of the state, thus overruling the principle of law laid down in the Hooppiania case. The fifth judge said "I do not repudiate the doctrine of correlative rights as applied in the Horne case," and added that, "The doctrine has its application chiefly to underground diffused and percolating water." This judge favored the doctrine of appropriation in the Wrathall case. Again, two judges said that they were not inclined to overrule the Hooppiania case at this time, while the fifth judge was silent on this point. One judge went so far as to say that if Wrathall's complaint were only that the *pressure* of his well had been affected, there would be no cause for action.

Another key case

Several days after handing down the decision in the Wrathall case, the Supreme Court handed down a decision in the Justesen-Olson case, and in so doing applied the doctrine of appropriation and prior rights. The facts in this case showed that Justesen had developed several artesian wells prior to 1903 on his own land, and that a subsequent pumped well drilled a few years ago by Olson on his own land resulted in lowering the pressure so that the Justesen wells failed to flow as they had before Olson started pumping his well. The information at the trial showed that both parties were securing water from the same water basin. In deciding the case the supreme court found in favor of the earlier artesian wells and held that Olson must stop pumping whenever the former wells did not flow the normal amount that they had flowed prior to the drilling of the latter well. In other words, the decision applied the doctrine of appropriation and prior rights and declared the water to be a public water supply, while the source had been generally recognized as coming from an artesian basin.

This decision was a divided opinion of the court. Three judges concurred in the ruling that the doctrine of appropriation and prior rights should apply, but one of these three judges stated that in supporting the application of the doctrine of appropriation and

prior rights to this case he did not repudiate the doctrine of correlative rights.

Situation confused

These two decisions have tended to upset the former theories on the underground waters in Utah, and as a result water users in general, and especially those using water from underground sources, do not know the doctrine which may be applied to the supply or whether they have any established rights to underground water. There is no clear-cut legal decision as to whether underground water in Utah is public water subject to appropriation and under the jurisdiction of the state engineer, or artesian water subject to the doctrine of correlative rights or percolating water and the property of the soil.

Salt Lake City vitally interested

Salt Lake City is vitally concerned in the question of underground water and its legal aspects. Prior to 1931 the city had practically no underground water holdings, and its water supply for municipal purposes was obtained from canyon streams through direct diversion and storage. In that year, however, the city found that these sources were inadequate, and in looking for an additional supply decided to enter the Murray artesian basin. Following up its decision the city purchased 800 acres of land in the basin, paying $300,000 for the land and acquiring the water rights to 137 artesian wells. Another $450,000 was spent in building pumping plants and pipe lines to deliver the water to the city distribution system. In addition to acquiring these wells, the city immediately filed on all of the water issuing from the wells, alleging in their applications that this water was coming from well-defined underground channels. All these applications were approved by the state engineer.

Last year, the city, feeling the effects of the drought on its water supply, embarked on a new underground water project by drilling seventeen wells along the edge of the Murray artesian basin and pumping from these wells 46 sec.-ft. of water for municipal use. These seventeen wells, completely equipped, cost the city only $254,000, compared to $750,000 for the Murray basin wells, the difference being due to the fact that in the later wells the city acquired no land in the artesian basin except for well sites, but filed on the water developed by these wells, alleging that the water came from well-defined underground channels.

Since the drilling and pumping of the seventeen wells the artesian-well owners in the Murray artesian basin have raised strenuous objections to the city's invasion of the artesian basin, claiming that it has materially affected the flow of their wells during the past summer. In fact, the well owners in the basin have organized to protect themselves against the aggressions of the city in the basin. Legal action is almost certain to be brought against the city by the prior well owners in the basin.

Should the court eventually hold that the Murray artesian basin is a public water supply, then the rights of Salt Lake City would date from the time of filing and be subject to all prior rights and liable for any interference to those prior rights. Should the court, in addition, hold that proper filings on the water of this basin should have been made since 1903, then a large number of prior artesian well rights will be invalidated as against the city's rights. Should the court further hold to the doctrine of appropriation but that filing on the water through the state engineer's office was not necessary to establish a right, then the city's rights would be subject to all the prior wells in the basin. Should the court hold to the doctrine of correlative rights, then the city has no land in the basin to cover the seventeen wells that it drilled last summer.

Proposed legislation

Legislation has been prepared and submitted to the state legislature, making all underground water public water, subject to appropriation and under control of the state engineer, with the provision that all existing rights to underground water be recognized. This bill, if it passes, will only apply to future appropriations of underground water and will not solve the controversies between existing rights to underground water. These must still be decided by the supreme court, and in view of the court's divided opinions in the two foregoing cases it is difficult to prophesy its probable attitude.

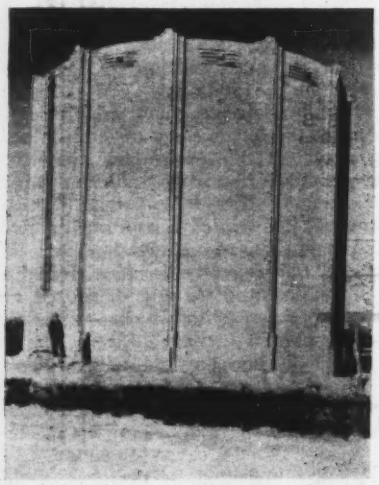
[EDITOR'S NOTE—The bill was signed by Governor Blood, March 22, as noted in this week's news.]

Prestressed Concrete Standpipe Built in Vertical Panels

By A. P. Kuranz
Superintendent, Board of Water Commissioners, Waukesha, Wis.

ONE of the first prestressed concrete water tanks to be built with vertical instead of horizontal construction joints has just been completed at Waukesha, Wis. The tank, of 675,750-gal. capacity, was added to the city's present storage system, to raise the capacity to about 1,000,000 gal. or enough to last 24 hours.

The site chosen for the new tank is on a high hill in a residential district overlooking the city and the surrounding country. It thus serves as a pressure as well as a storage tank. Also because of this location pleasing architectural treatment was considered essential. In cooperation with an architect (Martin White, of Waukesha) a simple yet monumental architectural design was developed which was used as a basis for inviting competitive structural designs.



CONCRETE WATER TANK at Waukesha, Wis., consisting of an interior prestressed shell and an outer architectural covering.

The circular wall is given architectural character by means of twelve pilasters, as shown in the photograph. When the tank was first proposed, surrounding property owners were violently opposed to it. However, today, because of the modern monumental design, residents consider the tank an asset to the neighborhood.

Tank design

In initiating this project, a number of firms were invited to bid and to submit detailed plans of the type of construction proposed. A minimum wall thickness of 10 in. was stipulated. Six bids were submitted, based on four different designs. The contract was given to the lowest bidder, who prepared a Hewett system design. This bid was $17,264.49 for the tank complete. The Hewett system makes use of circular steel bands, which are tightened around the outside of the main concrete wall, to prestress the concrete, eliminating cracks by keeping the concrete in compression under all conditions.

As built, the tank has an inside diameter of 50 ft., and the vertical wall

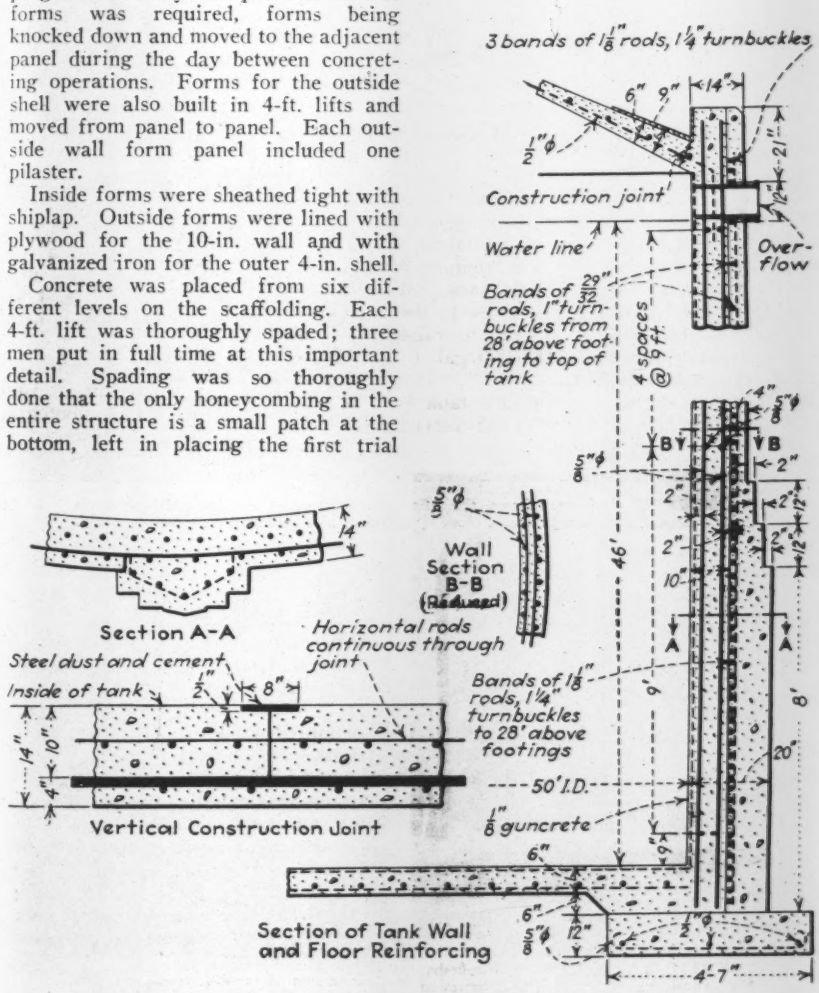
height is 46 ft. 1 in. to the overflow line. The roof is a reinforced-concrete dome, also a special Hewett design, using a ring of prestressed rods. The wall consists of a shell uniformly 10 in. thick from top to bottom, reinforced vertically in both faces, with ⅝-in. round rods 12 in. on centers. This shell is surrounded by horizontal rods 29/32 and 1⅛ in. in diameter, tightened by means of turnbuckles. Outside is a 4-in. protective shell of concrete, reinforced vertically by ⅜-in. round rods on 12-in. centers.

Construction methods

The inside shell was built in eight sections, and the outside shell in twelve sections. The joints in the outside shell fall at one edge of each of the twelve pilasters. Each of the eight inside wall sections was placed continuously from bottom to top in a one-day run of 8 to 12 hours, the fresh concrete laying tight against the section last built. A section was completed every day. The inside forms were built complete from bottom to top. Outside forms were laid up in sections 4 ft. high as concreting progressed. Only one panel of outside forms was required, forms being knocked down and moved to the adjacent panel during the day between concreting operations. Forms for the outside shell were also built in 4-ft. lifts and moved from panel to panel. Each outside wall form panel included one pilaster.

Inside forms were sheathed tight with shiplap. Outside forms were lined with plywood for the 10-in. wall and with galvanized iron for the outer 4-in. shell.

Concrete was placed from six different levels on the scaffolding. Each 4-ft. lift was thoroughly spaded; three men put in full time at this important detail. Spading was so thoroughly done that the only honeycombing in the entire structure is a small patch at the bottom, left in placing the first trial batch of concrete. No special attempt was made to secure bond at the vertical construction joints, but after all the forms had been removed an 8x½-in. recess left on the inside face at each vertical joint (see drawing) was given a priming coat of cement and steel dust and then filled with a mortar of cement and steel dust. The prestressing of the horizontal steel bands is depended upon to keep this construction joint from opening and leaking. The outer shell is not considered as adding anything to the watertightness of the tank. It is used simply for architectural effect and to protect the steel bands from corrosion. However, in making the construction joint in this exterior shell a definite attempt was made to break the bond between adjoining sections by using a heavy oil on the forms. Since the exterior shell could not be placed in compression by the horizontal bands, it was expected that there would be some shrinkage. By making a definite plane of cleavage at the construction joint, it is hoped to concentrate the shrinkage at this point. Thus unsightly cracks will be eliminated.



STRUCTURAL DETAILS of prestressed concrete water tank at Waukesha, Wis.

Concrete proportions were determined by trial batches, to give the desired workability with the aggregates available and the specified amount of water. Five and one-half gallons of water, gross, per sack of cement were used throughout. Two sizes of coarse aggregate were used in each mix. Proportions per sack of cement used in the floor and the 10-in. inner wall were: 224 lb. dry sand; 118 lb. ¼- to ⅝-in. gravel; 148 lb. ⅝- to 1½-in. gravel. For the outer 4-in. shell the proportions per sack of cement were: 245 lb. dry sand; 105 lb. ¼- to ⅝-in. gravel; 136 lb. ⅝- to 1½-in. gravel. The contract called for 544.6 cu.yd. of concrete, which was mixed and delivered in a truck mixer.

As a final operation both faces of the inside wall were washed with water and compressed air, to remove laitance and form oil. On the inside wall two coats of asphalt emulsion were applied under pressure, each coat being allowed to set 24 to 36 hours. A third coat, consisting of asphalt and sand, was applied, bringing the total thickness to ⅛ in. The tank was filled, to test for leaks before the outer shell was placed. In the entire tank to date, there is no evidence of leakage, not even a moist spot. The outside surface was sand-blasted, to remove form oil and give the desired finish.

Financing and insurance

The Waukesha board of water commissioners is a non-partisan commission elected by the council. The water department is self-supporting, and the project was paid for out of accrued earnings.

The extra water supply afforded by this tank changed the city from sixth to fifth-class fire rating, effecting a 5 per cent rate reduction for the business district—a saving which alone will equal the entire cost of the project in four years.

The tank was designed and built under the personal supervision of the writer. The structural features were developed by W. S. Hewett. The Pfeiffer Construction Co. was the contractor.

Grand Coulee Dam Engineers Move to New Town at Site

With progress on the government administration buildings and houses at the Grand Coulee project, on the Columbia River in Washington, advanced nearly to completion, the Bureau of Reclamation will transfer headquarters to the site from the town of Almira, where they have been located since the project work started. Spring weather in the mountains is beginning to increase the flow of the Columbia River at the dam site. For several weeks the river level was at high mark of last year's floodflow.

Airport Hangar Frame Built of Discarded T-Rails

By F. N. Wildish
Engineering Department, Illinois Emergency Relief Commission

HEEDING the Civil Works Administration's offer, partially to finance needed municipal projects, the city of Dixon, Ill., started and (under Illinois Emergency Relief Commission auspices) has finished an airplane hangar which makes economical use of local discarded materials, and at the same time contributes something unique to structural design and methods of construction.

The hangar, which is 91x100 ft. overall, is the central part of a project that includes a landing field approximately 2,600 ft. square, a restaurant and service buildings at the north central end of the field, the prevailing winds being due northeast. The field is 2 miles east of the center of Dixon, on U.S. Route 30.

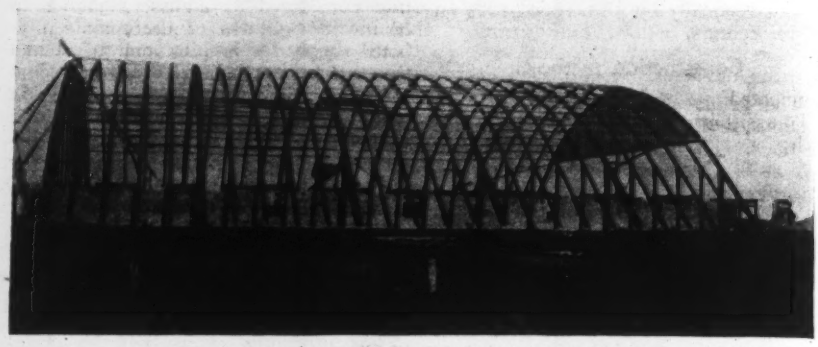
The structural frame of the hangar is composed of sets of 70-lb. T-rails (60 ft. long), anchored in concrete blocks at one end and bent in place over 10-ft. T-rail vertical struts, to form a parabolic arch, tied at the center. The rails were bent with the surface of the flange in a vertical position—that is, so that the axis of the rail itself forms a horizontal and not a vertical curve. Rails were placed on 4-ft. centers.

Each end rib and every fifth intermediate rib is composed of two rails, set flange to flange and bolted together. No roof purlins were needed in this type of construction. Nailing strips were set on the web of the rail and bolted through the rail; 2-in. dressed and matched lumber was nailed to these nailing strips, and over this a composition roof was laid. Each of the six double-member arch ribs was tied under the finished floor level by a 70-lb. rail section. This rail extended from an anchor in the arch anchor block through both the strut anchor blocks and then to an anchor in the other arch block. A concrete floor embedded these rails to a depth of about 30 in.

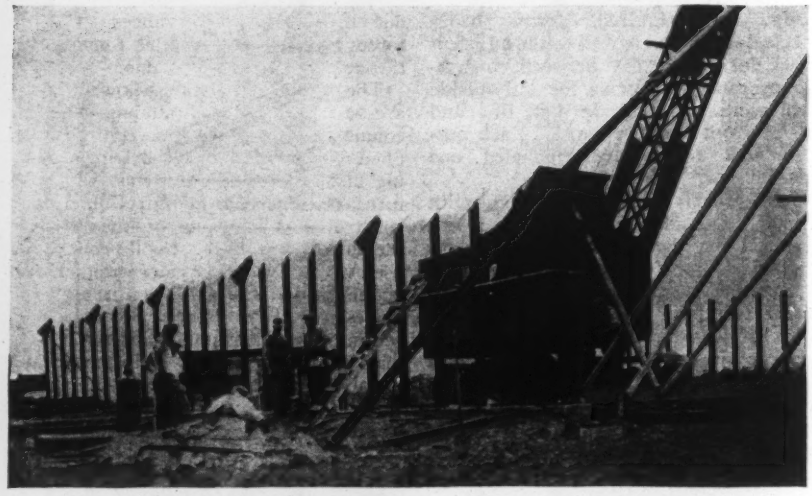
All the T-rails were donated to the city by the local street car company. This construction resulted in a 71-ft. clear interior width between side walls and a vertical height at the crown of the arch of slightly under 30 ft.

The first step in construction was the erecting of the vertical struts. These 70-lb. T-rails were anchored in a continuous concrete block 1½ ft. wide and 4½ ft. deep. The struts were embedded to a depth of about 3 ft. and extended to a height of 10 ft. above the ground line. The tops of the rails were burned off at an angle of approximately 45 deg., to accommodate the parabolic arch ribs which they supported.

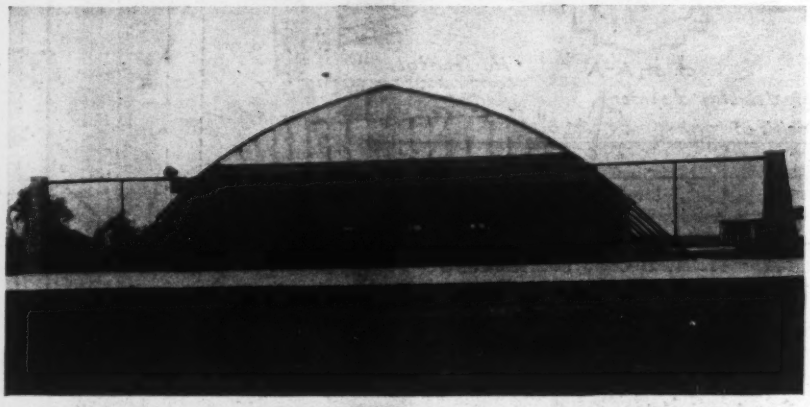
The next step consisted of pouring the continuous anchor block for the arch ribs. This block is 4½ ft. square, but was not built as a monolith, a section about 2x3 ft. on the hangar side of each of the two continuous blocks being omitted until the rails forming the arch ribs had been set in position under the ball of a rail extending the length of the hangar. This rail was partly embedded in the first section of the concrete block placed. With all arch rib and horizontal tie-rail connections made, the rest of the block was poured, the joints between



T-RAIL ARCH RIBS of airplane hangar at Dixon, Ill., which was built as an employment-relief project.



IN ERECTING the hangar frame, the rails were anchored at one end and bent down over a vertical strut to a parabolic curve by means of a chain block on a derrick boom.



COMPLETED HANGAR with brick walls erected on line of vertical struts, providing an enclosure 71 ft. wide and 30 ft. high at the apex of the arch and 10 ft. high at the eaves.

old and new concrete being strengthened by deformed bars.

Bending of arch ribs presented some difficulty. At first, bending on the ground before erection was tried, but there was such a variation in contours of the bent ribs that it was seen that an even roof surface could not be secured. The method which proved successful was found to lie in bending both halves of the arch at the same time with a chain block from the top of an erecting derrick. It was found, however, that the ends of the rails could not be brought entirely together, which resulted in a slightly sharper angle at the ridge of the structure than had been anticipated.

To assist in bending and to stiffen the arch ribs, a ¾-in. bar was fastened to the arch rib 1 ft. toward the crown from the strut connection, and to the crown end of the rib. When the rib had been bent a certain amount, the tierod was tightened by a turnbuckle, and the process was repeated until the rib had been bent to final shape. Tierods were fastened to the ribs by U-bolts. The original design for connecting the ribs to the struts called for three splice bars to be fastened through the webs of ribs and struts. The strength of this detail was deemed insufficient, so the connection was stiffened by a gusset plate bolted on the inside of the connection through both strut and rib webs and the splice bar.

The arch crown was closed by bolting, with ⅞-in. bolts, regular rail splice bars to the ends of the ribs. Splice bars were set each side of the rail web.

Light Sheet-Metal Wall Used for Airport Building

By Geo. D. Burr
Structural Engineer, Engineering Bureau, Public Utilities Commission, San Francisco, Calif.

DIFFICULTY in securing low-cost industrial-type buildings that are reasonably attractive in appearance led to the development, in a new machine-shop building at the San Francisco airport, of a type of light sheet-metal wall construction, employing a sanded paint finish resembling stucco in texture. The building is a two-story steel-frame structure resting on a concrete and pile foundation. The fenestration used was dictated by the type of building concerned and is not necessarily a part of the type of construction. The wall is designed for a 20-lb. per sq.ft. wind load and is also unusually safe from damage due to seismic disturbances. Lateral forces are carried to the foundations through the structural frame.

The reveals above and below the sash are structural girts spanning between columns. The space between girts is filled with vertically placed 1-in. sheathing fastened to nailing strips, which in turn are attached to the girts. In future construction, in place of the vertical sheathing, ¾-in. plywood will probably be used because of its greater rigidity and better insulating qualities; it also requires less labor in its application, presenting a smaller number of joints that may be hit by the nails fastening the metal covering. The steel sash is attached similarly to the sheathing.

The entire exterior is covered with 26-gage, flat galvanized steel sheets, with all joints set in mastic, and with all nail heads concealed. Narrow sheets were used, to eliminate buckling from temperature changes. After the sheet-metal work was completed, it was covered with two coats of portland cement paint, applied by spraying, the second coat of which was mixed with sand so that the resulting texture resembled stucco. The effectiveness of the sand-paint coat in hiding the joints and in eliminating light reflection from minor buckling of the sheets was notable.

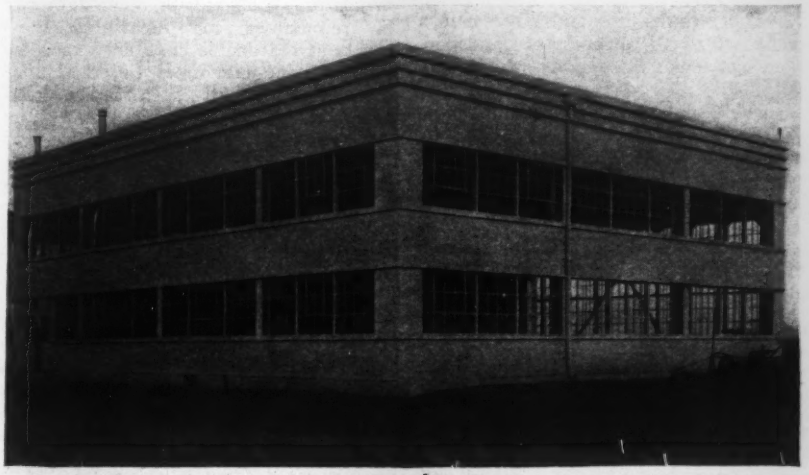
The costs of the construction on this building, erected under basic wage rates of about $1.33 per hour for skilled trades and $0.75 for unskilled labor, were as follows:

	Costs per Sq.Ft. of Wall		
	Material	Labor	Total
Girts, sheathing, nailing strips, hardware, plywood and interior painting	$0.079	$0.065	$0.144
Sheet metal and exterior painting	0.114	0.146	0.260
Total cost of wall without openings	$0.193	$0.211	$0.404

The above unit costs include the interior plywood wall finish, interior trim, and interior three-coat painting. If metal lath and plaster interior wall finish were used, the costs would vary accordingly.

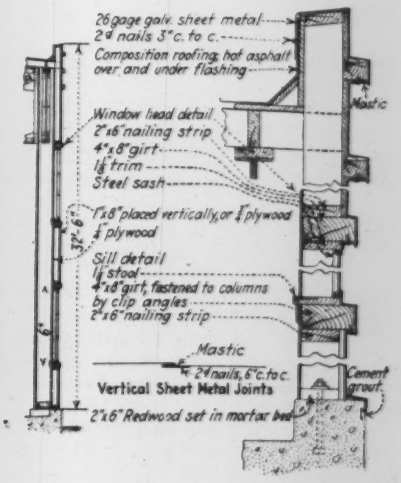
The advantages of this type of construction are:

1. It is exceptionally weathertight and resistant to leakage, even in the most exposed locations.
2. It is susceptible to many different styles of architectural treatment.
3. It is remarkably light, which reduces the dead load carried by the structural frame and the foundations, and also reduces the lateral forces from seismic disturbances, all of which in turn tend to reduce the cost of the building.
4. It is resistant to wind forces and



LIGHT GALVANIZED SHEETS comprise the wall facing of a new building at the San Francisco airport. Sanded and painted, the walls resemble stucco.

DETAILS of light wall construction, consisting of timber backing and a sheet-metal face.



not subject to serious damage or hazard from falling fragments during seismic disturbances.

5. It provides a wall insulated to any extent desired.

6. It is relatively inexpensive.

7. It is easily altered or repaired if damaged.

8. Wide horizontal ducts are provided in the wall in which conduits, pipes, etc., may be placed.

The building was designed by the writer under the direction of L. W. Stocker, chief civil engineer, and L. F. McAfee, chief engineer, of the engineering bureau, Public Utilities Commission, city and county of San Francisco.

Revised Plan of Flood Control for the Mississippi River

Includes payment for land rights for floodways, substitution for Boeuf floodway of a controlled floodway in the Tensas Valley, a new floodway in the Atchafalaya Basin and flood control of the St. Francis and Yazoo rivers

A REVISED PLAN of flood control for the lower Mississippi River that eliminates virtually all the registered objections to the official plan of 1928 (Jadwin plan) has been submitted to the committee on flood control of the House of Representatives by Major-General E. M. Markham, Chief of Engineers, U. S. A., on recommendations of a report by the Mississippi River Commission. The cost of the revised plan is estimated to be $245,000,000, not including remaining work still called for by the plan of 1928 and not eliminated by the revised plan. Including this remaining work, the total estimated cost is $313,000,000.

Main revisions of plan

The main elements of the revised plan, details of which are outlined in succeeding sections, are:

1. Payment for land rights for floodways and for restoration of drainage and for railway and road crossings interrupted by flood-control works. Such compensation was not provided for the two main floodways (Boeuf and Atchafalaya) in the 1928 plan, and a federal court decision enjoined construction of the Boeuf floodway, so that work in the Atchafalaya floodway was then limited to operations not involving land rights for flowage.

2. Provision of new main-river works, including a new floodway in the Tensas Basin to replace the old Boeuf Basin floodway; a new floodway in the Atchafalaya Basin, a new outlet near Morgan City for Atchafalaya drainage, and channel rectification of the Mississippi and the Atchafalaya rivers. Of these works channel rectification has been actively in progress under the 1928 plan as described in *Engineering News-Record*, June 22 to Aug. 10, 1933, and Aug. 8, 1934. The other works are new.

3. Provision for supplemental flood control by levees on the St. Francis and Little rivers in Arkansas and by reservoirs and levees on the Yazoo River in Mississippi.

The location of the planned new works in relation to the older levee system is indicated by the heavy solid black lines and areas on the accompanying map. The discussion of the plan and the recommendations for action as summarized from the reports of General Markham and the Mississippi River Commission develop the new projects as follows:

Revisions considered

General Markham states that the essential modifications of the 1928 project requiring consideration were as follows:

1. A revision of the plans for the floodway in the middle section of the river, to minimize the areas subject to overflow in great floods, and to assure positive operation of a floodway.

2. An elaboration of the plans for the Atchafalaya floodway, to reduce the amount of developed land subject to overflow in great floods, to assure positive operation, and to provide an adequate outlet to the Gulf for floodwaters.

3. The equitable and reasonable compensation of owners of lands in floodways under provisions which will protect the United States against unreasonable and excessive claims.

4. A revised program of works to increase the discharge capacity of the leveed river channels and for continuing the stabilization of its banks.

5. The relief of lands within areas protected by main Mississippi River levees from the consequences of recurring floods from hill tributaries.

Main-river works

In the upper section of the river from Cape Girardeau, Mo., to the mouth of the Arkansas River no changes in the 1928 plan are proposed. Changes are confined to the middle section from the mouth of the Arkansas River to the mouth of Old River and in the southern section from Old River to the Gulf of Mexico.

Middle Section—The plans provide for the abandonment of the Boeuf floodway and in lieu thereof the construction of a floodway extending from the vicinity of Eudora east of Macon Ridge. Control works are proposed at the head of this floodway, and a back protection is to extend north to the Arkansas River. The floodway is designed to withdraw from the main river all waters in excess of the safe capacity of the leveed channel. The control works are to be so designed that flow will not commence until the flood has reached a stage corresponding to 51 ft. on the Vicksburg gage (about 9 ft. below the levee grade), and will be stopped when the flood has receded to that stage. It is not intended that the spillway will be operated at all unless the predicted flood exceeds the safe capacity of the leveed channel in the main river.

The commission is of the opinion that provision for maintaining railroad and highway traffic across the floodway should be made, and that if reasonable cooperation is afforded by railroad and highway authorities these crossings should be provided at the expense of the United States. The commission also recommends that the United States acquire the right to improve flowage conditions in the floodway, to use it for flood discharge and to regulate this use; including the right to pass into the floodway at any stage any flow that can be safely carried in the natural or improved drainage channels. It recommends that the states or other authority be reimbursed for furnishing these rights at reasonable cost, not exceeding one and one-half times the total 1934 assessed value of such lands. Reimbursement by the United States for flowage rights south of the floodway levees is not contemplated, since the natural backwater area would in any event be overflowed. Furthermore, this backwater area will be substantially benefited by a second floodway proposed in the southern section.

The intake of the Eudora floodway is 120 miles by the present river course below the confluence of the Arkansas, and the present capacity of the leveed main-river channel is not sufficient to carry a great flood down to this proposed intake. Accordingly, the commission recommends that a black protection levee, of the same height as the levees on the east side of the river, be

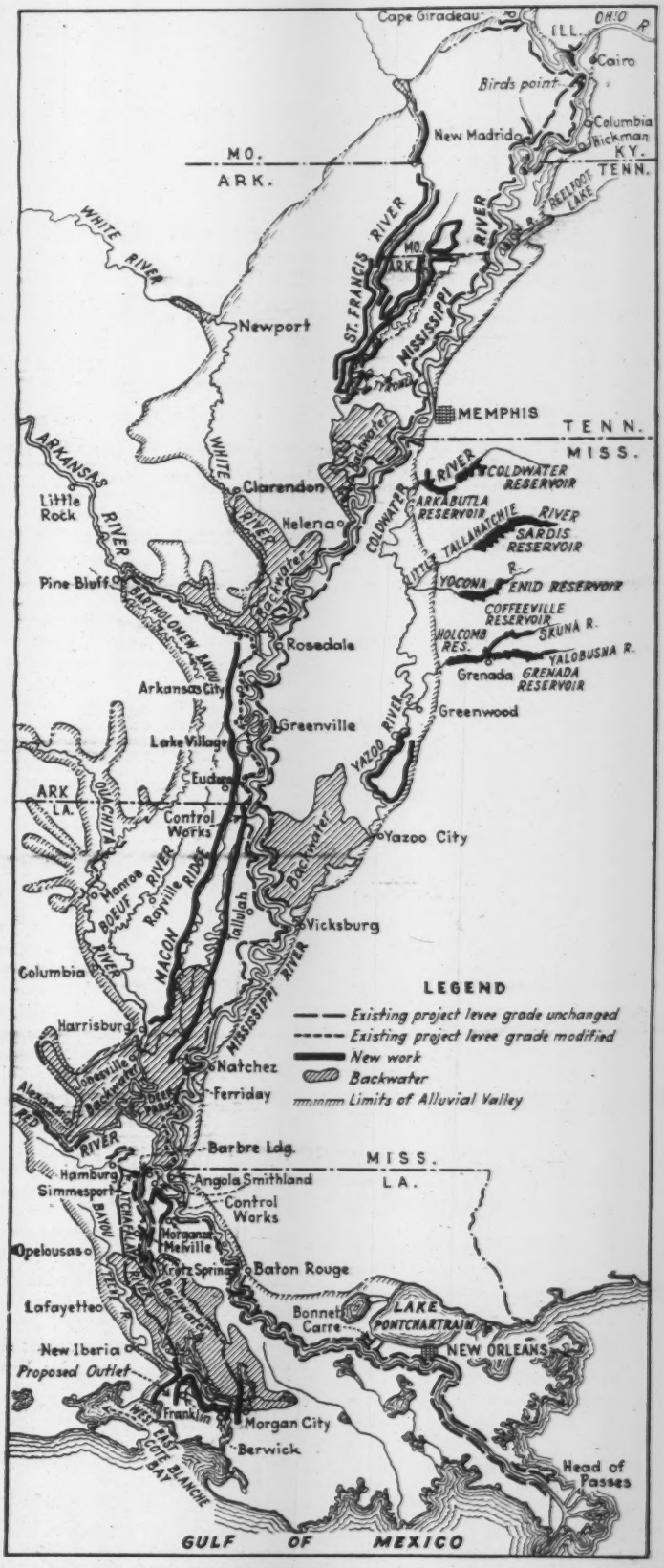
constructed as a northern extension of the western side levee of the floodway. It expresses the view that eventually this levee should be extended north to the Arkansas River in the vicinity of Yancopin, generally east of the railway. It should be constructed after the Eudora floodway is completed and when flowage rights and levee rights-of-way are provided by local interests under the same conditions of reimbursement as provided for the Eudora floodway. The present Mississippi River levee, between the head of the Eudora floodway and its northern junction with the back protection levee, is to be left at 1914 grade and section, except where it is to be raised and strengthened as part of a ring levee. The commission recommends that, at the request of local interests, control works may be installed in this levee at federal cost similar to those proposed at the head of the Eudora floodway.

The commission points out that the rectification of the Greenville Bends and the drawdown of the Eudora floodway, together with the gradual construction of reservoirs on Mississippi River tributaries, will substantially reduce flood heights in the section between the Arkansas and the Eudora floodway.

The other main levees under the present project should, it is stated, be completed except the lower part of the line between the Mississippi River and the Red River backwater. The estimated cost of completion is $3,000,000, and funds for the purpose are in hand.

General Markham concurs in the views of the commission that the plan proposed is the best yet devised to afford protection to the maximum area in the middle section of the river, provided that the proposed back protection levee extending from the head of the floodway to the Arkansas River is sufficiently set back to pass maximum floods without danger of overtopping either this levee or that on the east side of the river. Its exact location should be determined to meet this essential requirement.

Southern Section—In the southern section the commission proposes to substitute for the floodway east of the Atchafalaya River a floodway with a controlled intake on the Mississippi River north of Morganza. The control works are to be designed so that flow will not commence until the flood has reached a stage corresponding to 49 ft. on the Angola gage (11.5 ft. below levee grade), and will be stopped when the flood has receded to that stage. It is not to be operated at all unless the predicted flood exceeds the safe capacity of the leveed channels. The levees extending from this intake north along the main river and Old River to the head of the Atchafalaya are to be raised, to afford additional protection to the northern end of Point Coupe Parish. The land rights in the Morganza floodway are to be acquired, and roads and



NEW FLOOD-CONTROL WORKS on the lower Mississippi River as provided in the revised plan recommended by the Chief of Engineers, U. S. A., and the Mississippi River Commission.

railroad crossings will be provided as proposed for the Eudora floodway.

The Morganza floodway will afford, it is considered, positive assurance that the flood-discharge capacity of the Mississippi below will not be overtaxed. It should save the lands on the west side of the Atchafalaya from inundation except in extreme flood. The commission proposes, however, that these lands be included in those for which the acquisition of flowage is proposed, since they will continue to be jeopardized by the maintenance of a fuse plug levee at their head. The total cost of the works and flowage rights proposed, including the completion of the protecting levees to the final outlets into the Gulf, and the provision of the drainage works necessary, is estimated at $47,000,000.

The commission recommends the improvement of the Atchafalaya River to carry the maximum amount of floodwater without increase in stages. The work includes the enlargement of the openings of existing railroad and highway bridges across the Atchafalaya River and such alterations of existing crossings of this river as are deemed necessary to the execution of the plan. The estimated cost of these river improvements, including the alterations of crossings, is $15,000,000.

The 1928 plan provides for the escape to the Gulf of the waters of an extreme flood across the comparatively high ground to the east and west of Morgan City, the latter to be protected by a ring levee. In order that flood discharge to the Gulf may be more rapid, and the land referred to may be thus protected, the commission proposes an additional outlet west of Berwick consisting of a dredged channel and levees with necessary highway and railroad bridges and navigable connections with the Intracoastal Waterway. The estimated cost of this outlet is $8,000,000.

The completion of the main-river levees in this southern section with the project as modified is estimated at $1,000,000, for which funds are already in hand.

The total cost of the additional works proposed, including compensation to the land owners in the floodways, is $70,000,000.

Supplemental Work—The commission recommends that the program for the stabilization and maintenance of the channel of the Mississippi River, as provided in the present project, be continued and supplemented by additional work on cutoffs, dredging, etc., for the specific purpose of increasing its discharge capacity. It recommends the total authorization for river improvements and maintenance for a period of six years of $72,000,000.

The commission points out that reliable roads along or on the levees, in addition to those agreed upon in connection with floodways, would be of material, or even critical, assistance in the emergency of a high flood. It recommends the expenditure of $700,000 per year, or a total of $4,000,000 for a six-year period in the improvement of these levee roads.

Flood control on the St. Francis

A comprehensive plan for the control of the St. Francis River above the backwater of the Mississippi was made in 1929 and is published in House Document No. 159, 71st Congress, second session. The plan provides for enlarging and extending the existing levees, including the necessary diversion of floodwaters within the basin, for setting back certain levees to provide larger channels for the passage of floodwaters, and for enlarging by dredging the capacity of the Tyronza River, a tributary of the St. Francis. The estimated cost of the project, as set forth in that document, is $13,485,960. Subsequent to this report the local levees have suffered much additional damage because of bank caving and otherwise; and although extensive emergency repairs have been undertaken by the federal government under the provisions of Section 7 of the Flood Control Act of 1928, the cost of providing a complete and adequate flood-control system has been increased. This cost is now estimated at $16,000,000.

Flood control on the Yazoo

The Yazoo Basin is a large lowland area on the east side of the Mississippi River in the northwest part of the state of Mississippi. The Yazoo River drains this area and the adjacent hill region and enters the Mississippi at the foot of the basin near Vicksburg. A comprehensive report on the Yazoo River, including the control of its floods, is presented in House Document No. 198, 73d Congress, second session. The board of engineers for rivers and harbors, reviewing the report, found that the most suitable means for flood control lies in the construction of seven detention reservoirs on the hill streams, as set forth in that report, to provide essentially complete protection. The estimates of the cost of this reservoir plan are as follows:

Reservoirs:	
Construction cost	$28,231,000
Lands and damages	12,635,000
Railroad relocation	3,344,000
Highway relocation	1,440,000
Total	$45,650,000
Levees:	
Construction	$1,302,000
Rights-of-way	143,000
Drainage	2,650,000
Total	$49,745,000

The commission recommends the execution of this plan, at an estimated federal cost of $48,000,000, local interests to provide levee and ditch rights-of-way and highway alterations and damages.

Summary of costs

The cost to complete the total flood-control project as modified, in addition to funds on hand, is as follows:

Eudora floodway, including back levee to Arkansas River	$103,000,000
Atchafalaya Basin floodways	4,000,000
Improvement of Atchafayaya River and basin, including additional outlet west of Berwick	23,000,000
Main-river improvement and stabilization	72,000,000
Roads on or along levees	4,000,000
Flood control of St. Francis River	16,000,000
Flood control of Yazoo River by 7 reservoirs	48,000,000
Total	$313,000,000

Letters to the Editor

Rational Formula for Thickness of Flexible Pavements

Sir—It has been interesting to note the increasing attention given to the rational design of flexible pavements, a result undoubtedly of the large mileage of such surfaces constructed during recent years. Of recent comments two are of particular interest, one by Frank T. Sheets, consulting engineer of the Portland Cement Association, which appeared in your Jan. 17, 1935, issue, and the other by Mr. Fleming, of the same association in the issue of March 7.

It is not intended particularly to point out the erroneous conclusions which may be drawn from the use of these formulas, but rather to draw the distinction between the bituminous types and the non-bituminous types of flexible surfaces. While certain bituminous surfaces do have very definite flexural strength, in fact may be constructed to behave as rigid pavements, they are generally classified as of the flexible type, and it is indeed desirable that such pavements have a certain degree of flexibility to develop most completely the full inherent strength of a given subgrade condition. However, it is very obvious that there is a wide difference in the load-supporting capacity of a loose gravel surface, a loose macadam surface, a waterbound macadam surface or an asphaltic concrete surface. This difference is very largely the difference in resistance to shear, and in the laboratory the control of bituminous mixtures is measured by means of the stability test.

The formulas mentioned by the two authors are all derived from the principle that a load applied to an elastic surface

is transmitted at an angle of approximately 45 deg., and thus for the maintenance of equilibrium in any structure there must be a unit subgrade support greater than this load divided by the area of contact over which the transmission occurs. The difference between the several formulas noted is largely in the assumptions as to the angle of transmission of load, a subject upon which there had been relatively little research until recently.

There is a great difference in the reactions in a pavement under static and moving loads in regard to both the angle of transmission of load and the consequent subgrade reaction. Housel and Vokac, of Michigan, have contributed illuminating studies in this regard. They have shown that subgrade resistance is made up of two elements: perimeter shear and the pressure bulb; and that under certain conditions all the load may be taken by perimeter-shear resistance with even a negative reaction under the center of the load.

Therefore, while very soft subgrades unconfined might in theory require a great thickness of loose material, in fact they require only a modification to increase this shear resistance greatly. No competent engineer would think of building 20-in. thick bituminous pavements; he first develops a high subgrade resistance by either admixture to the soil itself or by placing a layer of cinders, sand, gravel or stone according to availability at low cost. The Calcium Chloride Association, in its recent publications, has set forth the art of constructing these earth-gravel concretes so as to develop maximum stability, and in so doing has performed a distinct service for the highway engineer.

Bituminous pavements are placed upon subgrades that have been so improved, and are of such thickness that they will distribute the load safely, and also will waterproof and protect the subgrade from abrasion. Any subgrade may be brought to a 25- to 50-lb. per sq.in. resistance with a reasonable expenditure, and thus the standard designed thickness for bituminous pavements ranges from 1- to 5-in. maximum, according to load conditions. The great economy of bituminous pavements lies in their ability to be used in conjunction with other flexible materials; and by their very flexibility to maintain intimate contact with each underlying layer, thus developing the maximum inherent strength of the combination, and yet by reason of the very small deformation that takes place in them, still maintaining smooth riding surfaces.

While any pavement will be destroyed under high impact stresses, with present-day conditions of both vehicle and road surface, an allowance of 50 per cent for impact is pure extravagance, and will lead to unnecessary road costs. All field observations indicate that it is far cheaper to repair the isolated areas of insufficient subgrade support. The moving-load reaction is such that in flexible pavement design impact may be disregarded, once subgrade stability has been provided, because there are no bridged areas.

It is of course correct to take into account the area of tire contact as increasing the area of subgrade over which the load is distributed. With the above factors considered, the writer has written the formula for flexible pavements as follows:

$$t = .564\sqrt{\frac{W}{S}} - e$$

wherein t = thickness of bituminous pavement in inches, W = unit wheel load in pounds, S = unit subgrade resistance in pounds per square inch, and e = radius of tire contact in inches.

With the present-day procedure in soil stabilization, the designed thickness of bituminous pavements may be shown as in the accompanying table for variable wheel loads.

PROPER PAVEMENT THICKNESSES

Wheel Load	Subgrade Resistance S in Lb. Per Sq.In. 25 $t=$	30 $t=$	35 $t=$	40 $t=$	45 $t=$	50 $t=$
2,000	2.3	1.8	1.5	1.2	1.0	0.7
4,000	2.9	2.4	1.8	1.4	1.1	0.8
6,000	3.8	3.2	2.5	2.0	1.6	1.3
8,000	4.6	3.7	3.0	2.6	2.0	1.6
10,000	5.3	4.3	3.6	2.9	2.4	2.0

BERNARD E. GRAY,
Chief Highway Engineer,
Asphalt Institute.

New York, N. Y.,
March 27, 1935.

Dowels for Dummy Joints

Sir—Your summary of Mr. Toms' paper "Dowels and Joint Fillers for Concrete Pavements" (*ENR*, Feb. 28, 1935, p. 318) is most interesting. It focuses attention on an important element in the design of concrete pavement very closely related to its structural integrity and its riding qualities.

There is no question about the need for load transference at through transverse expansion or contraction joints. At such joints there is a complete severance and separation of the concrete, and the slab edges so formed are certainly as much edges as are the marginal sides of the road where strengthening by increased slab thickness is quite generally recognized as necessary. Strengthening the joint edges by load transference to the adjoining slab is undoubtedly the simplest and most economical means of accomplishing this purpose.

With reference to dummy joints Mr. Toms is quoted as saying: "The cost of forming the weakened plane is almost negligible, the principal item of cost being dowels for load transfer." This raises a question as to why the dowels should be used at all. The following is a discussion of that point.

It is the essential characteristic of the dummy joint that beneath the groove which weakens the section the abutting faces of the concrete are of the very rough, irregular and interfitting type produced in any natural fracture in concrete. Projecting pieces of coarse aggregate in one slab nest into companion recesses in the other slab; the general trend of the crack is in a plane normal to the pavement surface, but warps and departures from this plane will cause first one side and then the other to overlap.

The width of the crack at the dummy joint will depend upon joint spacing. With a 15-ft. spacing normally the crack will be only half as wide as with a 30-ft. spacing.

The load transference at the dummy joint will depend upon the crack width and also—a feature apparently ignored in some researches on the subject—upon the size of coarse aggregate used in the concrete.

It is obvious that if there were no coarse aggregate and the concrete consisted of a mortar, a crack of very slight width would completely destroy load transference by aggregate interlock along the joint. It should be equally obvious that if the coarse aggregate is large sized, with the crack widths possible in a 15- or 20-ft. joint spacing, it is practically impossible not to get load transference. Under such conditions it seems wholly unnecessary to spend money for dowels.

Evidence on this point comes not from laboratory tests but from tests in service under actual field conditions under a wide diversity of subgrades and traffic and continuing for varying periods of time up to ten years or more along the Pacific Coast in Washington, Oregon and California. With 2- and 3-in. maximum sizes of aggregate and 15- or 20-ft. spacing, undoweled dummy joints work very satisfactorily, practically without distress: corner breaks are rare, and edge weakness does not develop.

Under conditions prevailing along the Pacific Coast dowels for dummy joints appear to be only an expensive and costly remedy for a non-existent disease. Their omission would allow the money expended for them to be put to beneficial uses.

H. F. FAULKNER,
Junior Engineer,
City Engineer's Dept.

Seattle, Wash.,
March 8, 1935.

Correction

In the letter by F. E. Turneaure, Dean, College of Mechanics and Engineering, University of Wisconsin, (*ENR*, Feb. 21, 1935, p. 295), commenting on continuous-beam design, the quantity l should in every case be the figure 1. For example, the denominator to equation 4 should be

$$1 + \frac{b_n + 1}{2}$$

and not

$$l + \frac{b_n + l}{2}$$

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A Pioneer

THOSE OF US who have grown up in an age which supplies every field of activity with at least one journal directed to discussions of its particular problems little realize how handicapped were our forefathers by the lack of such sources of mutual exchange of information and comment. For this reason it is difficult for us now to appreciate the value of what Henry C. Meyer did for the engineering profession, especially its sanitary engineering division, when in 1877 he founded the *Plumber & Sanitary Engineer* as a forum for the discussion of the sanitation questions of his day. House sanitation then was the principal problem in his chosen field, as the name indicates, but other problems also needed discussion, among them slum conditions. To this subject Mr. Meyer gave active attention, fostering the adoption of the first tenement-house law in New York State. Under his aggressive leadership the field of the *Plumber & Sanitary Engineer* expanded and the character of its content changed, changes that are reflected in changes in name to the *Sanitary Engineer,* then to *Engineering & Building Record,* and ultimately to *Engineering Record.* This is the name the paper bore when Mr. Meyer, after twenty-five years' active direction of its editorial and business policies, sold it to James H. McGraw. Both Mr. Meyer and Mr. McGraw saw in technical journalism something more than a reporting service; they looked upon it as an agency of leadership in a chosen field of activity. The accomplishments of these earlier days and the principles laid down still serve as a source of inspiration to those who have been privileged to carry on the work that they so ably started.

Silicosis Legislation

WHAT CAN BE DONE in providing sound, reasonable and equitable legislation governing compensation of silicosis cases has been demonstrated by a new law passed in North Carolina. Here silicosis and asbestosis are included among 25 specific occupational diseases listed as compensable under an amendment to the workmen's compensation act of 1929. Several desirable features of the amendment are worthy of mention. Industries subject to the hazard of silicosis will be designated by the industrial commission, and employers in those industries will be so notified. Medical examination by a competent board is required of all present and prospective employees in the designated industries. Compensation will be paid only in case of actual disablement or death. A humane feature is allowance of limited compensation for those found to be afflicted with silicosis but not actually disabled, until they can find work in non-hazardous occupations. Finally, and logically, the act applies only to exposures after the effective date of the act, eliminating the troublesome old-case considerations that have hampered enactment of similar legislation in the past. If the compensation insurance premium rates will now take into account the degree of protection offered by an employer, an incentive will be furnished for real protection and ultimate elimination of the silicosis hazard. North Carolina has done a good job with a perplexing problem.

Underground Water

STRONG EMPHASIS is laid on the uncertainty that surrounds the legal status of underground water by the Utah court decisions and the new water-rights law of that state reported in this issue (pp. 487 and 498). Experience has demonstrated that this uncertainty leads to reckless competition for groundwater, and in turn such competition ultimately results in loss to the whole community. The unsettled state of law appears to be due in part to a desire to fit the Western water-appropriation doctrine to groundwater, and in part to attempts at distinguishing between different conditions of underground water occurrence according to the nature of the flow—whether artesian or locally percolating, and whether flowing in a defined channel or merely moving by gravity along the groundwater slope. Since underground water is concealed from view, its true physical condition and source can be explored only very imperfectly, however, and distinction according to nature of flow is therefore more likely to be a source of trouble than an aid in preventing trouble. Further, however well or ill the appropriation doctrine may be adapted to surface waters, in application to groundwater it is unsound doctrine if it fosters competitive appropriation. Groundwater is rapidly becoming essential to community welfare, especially in the West. Early recognition should therefore be given to the conservation and equitable use of such water, the more so as the origin of both percolating and artesian waters is in no sense local. The problem is sufficiently complex to warrant joint consideration by a conference of qualified representatives of the Western states, convened to develop a uniform doctrine that would safeguard both private and public rights.

Difficulties in a New Art

PRESENT INDICATIONS are that the vibration of concrete is more of a specialized craft than it was assumed to be at the outset of its recent more general use. Specification of vibrated concrete for the extensive navigation dam and lock construction program under the direction of the army engineers, a year or more ago, proved particularly useful in developing evidence. Experience on these locks and dams has shown, as was to be expected, that different concrete compositions and consistencies and even different qualities of aggregate modify vibrating procedure. A less-expected lesson is that the timing, spacing and location of the vibrating action have a positive bearing on its effectiveness with different concretes. Vibration calls for craftsmanship, in short. It is in this thought that the comments on vibration contained in the article on Allegheny River Dam No. 3, elsewhere in this issue, should be interpreted. The quoted records of increased concrete strength gained by vibration are convincing proof that it has a vital place in the technique of concrete construction, but at the same time the story shows that much more needs to be known about vibration before it can take rank as a precise tool of the concrete constructor. Experience will produce this knowledge if it is studiously interpreted and recorded. The present situation is a challenge to perfect the technique of the operation and to crystallize it in guiding rules.

Mississippi Flood Control Reoriented

RARELY does a matter-of-fact engineering document involve such broad interests as does General Edward M. Markham's report and recommendations on the Mississippi River flood-control project. The document is important not alone because of the sensational cost increase proposed—from $318,000,000 to some $550,000,000—but also because it represents a virtual reorientation of the hotly debated Jadwin plan, adopted by Congress early in 1928. The reorientation goes so far as to create in effect a new plan, or at any rate to establish a policy from which the ultimate plan will evolve.

The 1928 plan, it will be recalled, was prepared under the President's instructions because the flood calamity of the previous year had given rise to a countrywide demand that the nation take over responsibility for controlling the Mississippi, the main drainage channel of half the United States. The President, however, arbitrarily set a limit of $300,000,000 for the cost to be incurred. The outcome was the Jadwin plan, with its cost just under the prescribed limit ($297,000,000, later raised to $318,000,000), though estimates by the Mississippi River Commission approached $700,000,000, while others placed the cost of an adequate system of control works still higher.

It is a reasonable guess that the official plan was cut to fit the cloth, and that its shortcomings and defects were largely due to that cause. That it could not provide complete protection at anything like its estimated cost was recognized almost from the first, though not always admitted. That it contained gross inequities appears from its ruthless fuseplug levee system as well as from the bitter litigation that was initiated when, at the time construction of the Boeuf bypass was about to begin, hundreds of farmers who would be penned between the bypass border levees saw themselves doomed to the terror of living on the bottom of a potential river bed. Not only did the plan take little account of either economics or equities but it limited flood protection narrowly to the main stem of the river instead of considering also the protection of basins and tributaries—which, of course, would be a vastly more formidable undertaking. Reservoir methods were rejected as too costly.

The weaknesses and defects of the plan were directly related to the arbitrary cost limitation. Had compensation for exposure of bypass lands to flood destruction been included, a quite different economic picture would have resulted; the Boeuf bypass might not have been adopted, and the entire plan would have been changed.

In the new plan the Boeuf controversy is eliminated; the bypass is to be shifted to the Tensas valley. But aside from improving many details of the plan as the result of more thorough study, and adding the important feature of a second delta flood outlet (at Morganza), the Markham recommendations recognize the principle of federal payment for the entire project and the proper claim of the valley residents to equitable treatment, a principle that was clearly intended by Congress but that was quite imperfectly embodied in the original plan. The change is reflected in the recommendation to pay for flood easements over the bypass channel lands, in the inclusion of drainage for the lands back of the bypass levees, and in the discarding of fuseplug levees in favor of controlled spillways. By determining on this course the report disregards the arbitrary cost limitation set by the 1927 administration and squarely takes account of the physical facts of the problem with a view to providing a complete solution. Most briefly stated, the reorientation brought about by the present report consists in taking account of the full magnitude of the group of flood problems in the Mississippi and shaping the practical plan accordingly.

For this reason, the extension of scope that lies in the proposal to build protective works in two of the flood basins, in order to safeguard them against floods caused by their own tributaries, is even more notable than the modifications of the plan for the main stem of the river. Ultimate extension of the project to include consideration of the whole flood problem of the Mississippi drainage has been certain ever since 1927, and the intent of Congress to work in this direction has repeatedly been indicated. While inclusion of basin flood protection in the Markham recommendations may have been motivated in part by this year's extensive floods in the St. Francis and Yazoo basins, it also reflects a conviction that the basin floods are an integral part of the river flood problem as a whole, and that, while the lower portions of the basins must forever be dedicated to backwater servitude, it is sound engineering to plan for security of their upper portions.

Inclusion of reservoirs in the basin protection plans has the further significance of representing definite recognition, at least in germ, of reservoir control as an element in Mississippi River flood protection. It is quite likely to prove a beginning of what must ultimately find application to a large part of the Mississippi system, namely a widespread series of reservoirs. The needs of local protection will undoubtedly lead to the construction of a great many such reservoirs in the course of time; and, since floods in the main river are merely the resultant of floodflows in its tributaries, such regulation of headwater streams will in due course transform the river flood problem also. Storage to protect the basins, as well as any headwater storage such as that to be provided by the Muskingum and Tygart dams, is therefore a beginning toward thoroughgoing change in Mississippi River control.

Because of the virtual certainty that the original plan could not long remain unmodified, it has been under constant reinvestigation ever since the construction period began. The Vicksburg Experiment Station's studies and the full-scale trials of cutoffs and channel improvement by the Mississippi River Commission under General H. B. Ferguson were important parts of this work. During the past two years, moreover, the work of the National Resources Board has directed strong attention to the necessity for comprehensive stream planning and to attendant water-conservation purposes. In a broad sense it may be proper to look on the Markham report as the final resultant of these influences.

In all respects the report takes rank as a constructive move toward complete and satisfactory solution of the flood problem in the alluvial valley of the great river. It may not be the final step. Continuing active study is both appropriate and necessary in so great a problem. The final evolution may include extensive use of reservoir control, or, on the contrary, it may definitely contemplate maintenance of a large variation in flow between low water and flood time, according to what the further development of knowledge may indicate. But the plan now recommended, while providing securely for present conditions, establishes a firm basis for all further development of Mississippi flood control.

CURRENT NEWS

Power Survey Report Shows Need for New Construction

AN ACUTE shortage of power generating capacity is in prospect upon the resumption of normal business activity, according to the first interim report of the National Power Survey made by the Federal Power Commission. In transmitting the report to Congress, vice-chairman Basil Manly, commissioner in charge of the survey, stated that much has been said during the past five years about a supposed surplus of electric generating capacity. This, he said, until recently had been true enough if the situation were viewed from the short-range standpoint of relation between existing capacity and immediate demand during a period of depression, but that in recent months the use of power has been rapidly increasing until in many districts the demand is nearing the limit of available efficient generating capacity.

"It takes two years to plan and construct a large modern steam plant and from two to seven years to plan and build a large water power project. It is the part of wisdom, therefore, to determine at this time what new generating plants and transmission lines will be required to meet the demand that will be created by resumption of normal industrial activity and where they should be located to supply the demand most efficiently and economically. It is time also to take stock of existing private and public plants and ascertian which of them are inefficient or have become obsolete."

The survey was undertaken by the Federal Power Commission under authority of an executive order issued by the President on Aug. 19, 1933. Its staff of engineers and statisticians is working under Thomas R. Tate, director, and Lester S. Ready, chief consulting engineer.

Findings of the survey

The principal findings of the interim report of the National Power Survey have been summarized by the Federal Power Commission in the following:

1. The use of electricity for domestic purposes as well as in certain branches of industry has grown at such a rate during the depression that, upon a resumption of normal industrial activity, the demand for power will be at least 4,000,000 kw. in excess of that which existed in 1929. This is the equivalent to the capacity of some fifty large generating stations.

2. Very little new generating capacity has been constructed by the privately-owned utilities since 1930. As a result the capacity of existing plants is 2,325,000 kw. less than the demand that will exist for power upon a resumption of pre-depression industrial activity, assuming maintenance of normal reserve capacity.

3. This shortage is being rapidly accentuated by the obsolescence of plants which would have been replaced under normal con-

(Continued on p. 502)

Project Engineer Named for Large PWA Jobs in New York

Public Works Administrator Ickes has announced the appointment of Wharton Green as resident project engineer for the Triborough Bridge and the Midtown Hudson Tunnel in New York City. Mr. Green will relieve Arthur S. Tuttle, who as PWA state engineer for New York served also as project engineer for these two projects. This will allow Mr. Tuttle more time to use in his capacity as president of the American Society of Civil Engineers. Mr. Tuttle offered to resign as state engineer but was asked by Mr. Ickes to remain. Mr. Green formerly was special agent in charge for the PWA Division of Investigations with headquarters in New York City. He had general supervision of investigations in New York, Pennsylvania, New Jersey, Delaware and Maryland. Before coming to PWA he was actively engaged in the construction business for many years.

New Underground Water Law In Utah Signed by Governor

Governor Blood of Utah has approved a new underground water law for that state which, primarily, amends the existing law relating to water rights by enlarging the definition of public water to include not only water flowing above or under the ground in known channels as has been the case in the past, but to cover all waters in the state whether above or under the ground.

The new law places all waters in the same class, makes it property of the public, subject to existing rights, and provides for administration and control by the state engineer, who is given power to order capping or plugging of wells to prevent waste or pollution of any underground water. New or additional rights to the use of any water in the state, whether above or below the ground, may hereafter be acquired only by application to the state engineer. (See p. 487 of this issue.)

Salt Lake City's underground water development in the Murray artesian basin has been challenged in a complaint filed against the city by the officers and members of the Salt Lake County Water Conservation Association. The complaint has been taken to the third district court for adjudication.

The complaint, which also names T. H. Humpherys, Utah state engineer, as defendant, seeks a review of the decision of the state engineer on January 10, which permitted Salt Lake City appropriation of ten second-feet of water from an underground channel described in the city's application as the Hinnen under channel. There is no such defined underground natural channel, the complaint declares, and the percolating waters are not subject to appropriation. Further the complainant adds that the city be enjoined from the alleged unlawful diversion of these waters and that the title to the waters be quieted in the plaintiffs and others similarly situated.

Administration of Work Relief Act Still in Doubt

(Washington Correspondence)

PRESIDENT ROOSEVELT'S plan for administration of the work relief program is still in doubt but it is believed that it contemplates a central board of which he will be the active head and Harry Hopkins, chief of the FERA, the executive officer. Hopkins has the driving force on which the President will rely to give the four billion work program a quick start. By avoiding the fatal delays in 1933 which crippled the public works program from its inception the Administration is hopeful that its new drive will stimulate business as well as provide temporary employment for 3,500,000 employable persons now carried on direct relief rolls or supported by various forms of work relief.

Related types of work embraced by the new program will be grouped for supervision by various members of the board. The board presumably will include Rexford G. Tugwell, Under-Secretary of Agriculture, Rear Admiral Christian J. Peoples, director of the Procurement Division of the Treasury Department, Harold Ickes, Administrator of PWA, Robert Fechner, director of Emergency Conservation Work (CCC), and Robert E. Wood, president of Sears, Roebuck & Co., as an advisory member representing business interests.

President Roosevelt has not yet indicated the extent to which public works of engineering character will figure in the new program, or whether he will continue to use established procedure in PWA for handling non-federal projects. Administrator Ickes has sought to maintain emphasis on public works in the hope that the President will include heavy construction as a major feature of the new program. Whether the course taken by the President in formulating the new program without the help of the PWA administrator implies subordination of public works to projects of a simple made-work character is a question on which the construction industry must wait for an answer.

The results of a nation-wide survey of potential public works conducted by the PWA now total more than nineteen billion dollars, but this merely represents an effort made by Mr. Ickes to keep the PWA in the limelight when it became evident to him that the President planned to set up the work program on a much broader basis. Such projects have no standing in PWA.

PWA projects awaiting funds

Applications for approximately $600,000,000 in non-federal projects now pending in PWA are probably sound. These have been sifted from a much larger volume. Applications for about $100,000,000 have been processed for allotment as soon as new money becomes available.

The allocation of $900,000,000 made by

Enlarged Conservation Program Proposed for Muskingum Valley

A three-way expansion program in connection with Ohio's Muskingum Valley Conservancy District project and involving an additional expenditure of about $16,000,000 has been placed before federal officials in Washington for consideration. The plans involve the building of 60 secondary reservoirs to supplement the 14 major reservoirs already approved; an increase in the number of civilian conservation corps camps in the district to aid in the development of several smaller reservoirs, and plans for the extension of the Salt Creek soil erosion program to cover all 16 counties in the district.

Ontario Acts To Void Contract For Power Generated in Quebec

Contracts previously entered into by the Hydro-Electric Power Commission of Ontario for the purchase of power from private companies in Quebec are declared illegal, void and unenforceable in a bill introduced into the Ontario legislature on April 1 by the provincial government. Introduction of legislation to void the contracts is the outgrowth of a long and heated controversy over acts of the commissioners who were in charge of the activities of the Hydro-Electric Power Commission before the recent change of government in Ontario. The contract provided for the delivery of power, chiefly from power plants on the Gatineau River and from the Beauharnois plant on the St. Lawrence River, both in the province of Quebec.

The government maintains that the contracts were entered into without regard to the provisions of the act setting up the power commission or the British North America act.

Works Relief Bill Returned by House to Conference Committee

Acting on instructions from President Roosevelt, administration leaders forced the House to send the works relief bill back to conference committee on April 1 for elimination of the amendments requiring Senate confirmation of appointments and confining the $900,000,000 non-federal allocation to projects on which one-third of the cost is expended on direct labor.

In an exchange of correspondence described by Secretary Ickes as "an honest difference of opinion" Senator Glass, in charge of the bill in the Senate, defended the labor amendment by explaining that "direct work" means actual work on the project but not necessarily at the site. The conference committee, he said, had been assured by Comptroller General McCarl that any work done in connection with a project, such as the building of machinery, would be construed as direct work. Doubt was expressed by Secretary Ickes whether the courts would sustain such a definition and he asserted that it would be impossible to administer non-federal projects under the limitation. The extent of direct labor, he said, is a question of fact in each case which cannot be determined until a project is completed. PWA might go ahead on a project and then have the Comptroller refuse payment.

Long delay in enactment of the bill was scored by Secretary Ickes. He said that the PWA is ready to let go projects totaling $100,000,000 right away and could "open up the throttle" on a lot more. "The difficulty is," he said, "we don't know in what form the bill will go through. There is no use in our working on new projects. The money might have to be given away."

Inclusion of cost of materials as part of the expenditure for direct labor would remove some objection to the amendment, Ickes said, but would create a lot of difficulty in administration. Though displeased at the action of the House, Senator Glass has stated that the Senate conferees would continue to serve.

Arizona Enacts Law To Handicap Parker Dam

Governor Moeur, of Arizona, has signed a bill prohibiting building of an obstruction across a navigable stream within Arizona's jurisdiction without consent of the Governor and Attorney General. The Governor has expressed the belief that he can again halt construction of the Parker diversion dam in the Colorado River.

Boston Contractor Loses Suit To Recover Extra Work Costs

Because the Crane Construction Company of Boston, Mass., relied upon certain letters and an oral understanding instead of following the terms of its contract with the state in regard to extra work, the full bench of the Massachusetts Supreme Court ruled on March 26 that the Crane organization could not recover $37,304 extra costs incurred in blasting and excavating at the site of the Metropolitan State Hospital in Waltham. It appeared that the state's architect obtained through letter inquiry an extra work price of $7 per cu.yd. for open rock excavation, notified the contractor that the quotation had been approved by the state department concerned, and agreed to meet a representative of the contractor at the site. At the meeting a representative of the architect verbally authorized the prosecution of the extra work, which the lower and higher court found was carried out in good faith.

The contract for the job, involving $239,500, included provision for a written statement to the architect of the nature of the extra work performed within one week after beginning it, and for the filing of an itemized statement with the architect by the following tenth of the month. The court ruled that the letters did not constitute a written authorization to perform the extra work and that the contractor must suffer the hardship of his failure to secure such authorization and make the notifications required in the contract.

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the bill for non-federal projects may be raised or reduced to almost any figure the President wishes as he is authorized to transfer up to 20 per cent of the total from one allocation to another. As the same latitude applies to all other statutory allocations for roads and grade crossing elimination, river and harbor works, irrigation, sanitation, etc., the President may set up a program that includes very little construction of a permanent character either in the first instance or subsequently should it become apparent that the four billion dollar fund is not absorbing the unemployed rapidly enough to become an effective instrument of recovery.

Mr. Ickes hopes that, apart from any new money that PWA may receive, the President will restore the $310,000,000 of public works funds that were transferred to FERA for direct relief. Admittedly uninformed concerning the new program, the PWA administrator still has the courage of his convictions. His cautious policy of administration has discounted public works as a means of generating recovery but he sincerely believes that money spent otherwise will be wasted. "I don't know any other way it can be done," he said the other day. "I believe in all the speed that can be put on consistent with safeguards. I don't believe in scattering it. As much money as possible ought to be put out in self-liquidating projects. Of course, there has to be some inquiry to determine whether a project is self-liquidating."

CANTILEVER ARMS JOIN OVER MISSISSIPPI RIVER AT NEW ORLEANS

Last week, just before this picture was taken, the final member at the center of the 790-ft. cantilever span of the new Mississippi River bridge at New Orleans was erected, placing this important structure out of danger from any spring floods. There remains to be erected one-half of a 530-ft. through-truss span (left edge of picture) and an adjacent 330-ft. deck truss span before all steel will be in place. This work should be completed next month ready for deck paving. The bridge, which is 23,000 ft. long with its viaduct approaches will serve both highway and railway traffic *(ENR,* **Aug. 17, 1933, p. 190, and May 10, 1934, p. 589). The approaches are completed.**

The cantilever bridge, just closed, was erected by guy derricks, which started at the two center piers and worked both ways. The American Bridge Co. has the contract.

Financed by an RFC loan, the $13,000,000 bridge is owned jointly by the state of Louisiana and the Public Belt Railroad Commission. It was designed and erection is being supervised by Modjeski, Masters & Case, consulting engineers, New York, for whom C. Glennon Melville is resident engineer.

IN WASHINGTON

By Paul Wooton
Washington Correspondent

After an unusual amount of pulling and hauling the work-relief bill comes through with few additional limitations on the discretion of the President. The important thing is that $4,000,000,000 is about to be made available for the purpose of supplying direct employment to 3,500,000 persons. Having learned its lesson from past experience with public works, the Administration is prepared to do whatever is necessary to attain velocity in the expenditure of this great sum. It realizes that if the pump is to be primed the handle must be worked vigorously.

Unexpectedly strong opposition has arisen to the proposal of the Chief of Engineers and the Mississippi River Commission to divert excess Mississippi River flood waters at Eudora in northeastern Louisiana. Land owners and the parishes involved have acquainted the Flood Control Committee of the House that they will resist to the last legal ditch any such use of their lands.

Testifying before the Flood Control Committee of the House, Chief of Engineers Edward M. Markam revealed that the proposed west bank diversion from the Mississippi at Eudora, La., would be constructed so as to carry as much as 1,500,000 second-feet in the event of a super flood.

The Supreme Court of the United States has refused to dismiss the suit brought by the state of Nebraska against the State of Wyoming as requested by Wyoming. The suit involves the ascertainment of the equitable apportionment of the waters of the North Platte River as between the two states. An injunction against further use of water for the Casper-Alcova and Seminole reservoir projects is sought.

President Acts to Offset Let-Down in Code Operations

In a letter to Donald Richberg, chairman of the National Industrial Recovery Board, President Roosevelt on March 26 requested that the full power of NRA be exerted to obtain compliance with code requirements. Asserting that there is no excuse whatsoever at the present time for members of industry who have sponsored and are subject to these codes not to give them wholehearted support, he called upon the Department of Justice to take prompt and vigorous action to prevent or punish violations. In an effort to halt the very noticeable letdown in industry cooperation and compliance as a result of the uncertainty as to extension of the Recovery Act, the President also suggested that the Recovery Board bring the facts of this situation to the attention of Congress so that it may understand the importance of accelerating action on its decisions.

State Recovery Acts

The uncertainty of court decisions as to the constitutionality of certain provisions of the Recovery Act, and therefore of the codes of fair competition, brought about a program to have the several states of the Union enact state NRA acts that would regulate intra-state activities should the national act be declared ineffective in some respects. The latest reports indicate that 16 states now have recovery acts paralleling the national act, as follows:

California	Oregon
Colorado	South Carolina
Idaho	Utah
Illinois	Virginia
Indiana	Washington
New Mexico	West Virginia
New York	Wisconsin
Ohio	Wyoming

Four others have passed legislation to aid the operation of the NIRA, but without complete enforcement laws, three of which suspended the state anti-trust laws. A state act in New Jersey was abolished by the Governor last fall, but a new bill has been introduced into the present session of legislature. Bills also have been introduced into the legislatures of 22 other states.

It is specified in the model draft for state NRA acts that invitations to bidders shall contain a provision to the effect that no bid shall be considered unless accompanied by a compliance certificate executed by the bidder. This provision is similar to the content of Executive Order No. 6646 issued by President Roosevelt on March 14, 1934, under which similar certificates are required on projects employing federal funds, but omits the provision in this executive order whereunder contracts already awarded may be cancelled when noncompliance with applicable codes is proven.

Delaware County Roads Transferred to State

Effective July 1 all county roadways and bridges in the State of Delaware will be transferred from the Levy Court Commissions to the State Highway Department. At the same time the Levy Courts will discontinue the levying and collection of taxes for county road purposes.

Another Damage Suit Begun Over Abandoned San Gabriel Dam

The contractors on the abandoned San Gabriel Dam of the Los Angeles Flood Control District, Fisher-Ross-MacDonald & Kahn, Inc., on March 20, filed a claim for $4,290,459 against the district. The suit is a counter-claim to the district's suit against the contractor for $830,766, the amount of the district's settlement with the contractor following the abandonment of the work, plus $1,846,167, the amount paid the contractor for excavation work (*ENR*, Sept. 14, 1933, p. 332).

J. Waldo Smith To Be Honored By Tablet at Ashokan Reservoir

The tower on Winchell Hill at Ashokan Reservoir 30 ft. square and 40 ft. high, originally built as a triangulation tower of concrete masonry faced with native blue stone, is to be reconstructed and designated as the J. Waldo Smith Memorial Tower. The project is being carried out by Mr. Smith's friends with the cooperation of the Department of Water Supply, Gas and Electricity of the City of New York.

A large panel of stone suitably framed with like material is to be placed on the southwest face of the tower, and the small tablet now on the face of the tower giving the names of the chief engineers and of the consulting engineers is to be moved to the opposite face. The work is estimated to cost about $5,000, including the cost of maintenance. Friends who wish to participate are asked to send their remittances to John M. Carroll, Room 1104, 346 Broadway, New York City.

North Carolina and New York Include Silicosis in Compensation Acts

SILICOSIS and other occupational diseases were made compensable in North Carolina and New York by amendments to the State Workmen's Compensation Act passed recently by the legislature. A New York amendment which becomes effective September 1, 1935, includes "all occupational diseases." It does not apply to any case of occupational disease in which the last injurious exposure to the hazard of the disease occurred prior to September 1, 1935. There is no statement in the act defining an occupational disease, nor does it contain any provision for determining the degree of disability or responsibility.

The New York legislature also passed a bill designed to eliminate medical abuses in workmen's compensation cases. This new law provides that an injured employee, and not his employer, has a right to select his physician. The Industrial Commissioner is directed to establish a schedule of minimum charges and fees for such medical treatment and care. The bill also provides for licensing of compensation medical bureaus and the revocation of a license if the bureau resorts to corrupt or unethical practices.

The North Carolina amendment now in effect makes definite provision for the determination of the degree of disability suffered by an employee afflicted by any one or more of 25 occupational diseases specifically named. Silicosis and asbestosis are included in this list. Features of the amendment are abstracted in the following:

No compensation is payable for asbestosis or silicosis if the employee, at the time of entering into the employment of the employer by whom compensation would otherwise be payable, falsely represented himself in writing as not having previously been disabled or laid off because of asbestosis or silicosis. Compensation is payable only in the event of becoming actually incapacitated, because of an occupational disease, from performing normal labor in the last occupation in which remuneratively employed.

The provisions of the act apply only to cases of occupational disease in which the last exposure in an occupation subject to the hazards of such disease occurred on or after the effective date. In any case where compensation is payable for an occupational disease, the employer in whose employment the employee was last injuriously exposed to the hazards of such disease, and the insurance carrier, if any, which was on the risk when the employee was so last exposed are liable.

An employer is not liable for any compensation for an occupational disease unless disablement or death results within three years after the last exposure to such disease, or in case of death, unless death follows continuous disability from such

disease, commencing within the period of three years, and results within seven years after such last exposure.

When authorized by the Industrial Commission after consulting with the Advisory Medical Committee, the employer must provide reasonable medical or other treatment for such time as in the judgment of the Industrial Commission will tend to lessen the period of disability or provide needed relief, not to exceed a period of three years nor a cost of more than $334 in any one year.

The Industrial Commission will designate by order each industry found subject to any such hazard of silicosis or asbestosis and will notify the employer therein and compulsory examination of employees and prospective employees will be required. The Industrial Commission will make inspections of employment and keep a record of all employment subjecting employees to the hazards of silicosis or asbestosis and will notify the employer in any case where such hazards exist.

Where an employee, though not actually disabled, is found to be affected by asbestosis or silicosis and if such employee would be benefited by changing occupation, and if the disease has progressed to such a degree as to make it hazardous for the employee to continue in his employment, he will be paid compensation as for a temporary total or partial disability as the case may be, until he can obtain employment in some other occupation, compensation being limited to 20 weeks for an employee without dependents and 40 weeks for an employee with dependents. If the forced change of occupation requires that the employee be given special training in order to properly readjust himself, he will be paid for such training and incidental traveling and living expenses up to $300 for an employee without dependents, and $500 in the case of employee with dependents. If an employee has been compensated for an occupational disease and thereafter engages in any occupation which exposes him to the hazards of silicosis or asbestosis without obtaining the written approval of the Industrial Commission, he forfeits all right to future compensation.

Silicosis is defined as the characteristic fibrotic condition of the lungs caused by the inhalation of dust of silica or silicate. Asbestosis is a characteristic fibrotic condition of the lungs caused by the inhalation of asbestos dust.

Compensation will not be payable for silicosis or asbestosis unless the employee has been exposed to the inhalation of dust of silica or silicate or asbestos dust in the employment for a period of not less than two years in the state, provided no part of such period of two years was more than ten years prior to the last exposure. In case of disablement or death from silicosis and of asbestosis, complicated with tuberculosis of the lungs, compensation is payable as provided, except that the rate of payment may be reduced one-sixth.

The amendment provides for definite notification of the Industrial Commission in cases of manifestation of silicosis or asbestosis or death resulting therefrom, and also provides for inspection and examination. The inspections and examinations will be paid for out of a general fund, excepting that if sufficient funds are not available, employers will be liable to assessment.

New Maryland Law Authorizes Chesapeake Bay Bridge

The Maryland legislature on March 29 passed a measure authorizing the construction of a bridge from a point near Baltimore City across Chesapeake Bay to the eastern shore. The measure provides for a commission of three men who will be empowered to borrow $11,720,000 with which to purchase the Claiborne-Annapolis Ferry Co. and construct the bridge. Any borrowings of the commission shall not create or constitute an indebtedness of the state of Maryland or any political subdivision, but only of the bridge authority, according to the law. Whether the bridge is built or not, the commission is authorized to acquire the ferry company for $1,200,000 in 15-year 3 per cent bonds.

Henry C. Meyer, Founder of Engineering Record, Dies

Henry C. Meyer, founder of one of the papers combined in 1917 to form Engineering News-Record, died at his home in Montclair, N. J., on March 27, age 91 years.

Mr. Meyer was born in New York City in 1844, studied in private schools and after distinguished service in the Civil War, for which he was awarded the Congressional Medal of Honor, being discharged for disability, returned to New York and entered the employ of the New York Shot & Lead Co., manufacturers of shot and lead pipes. Subsequently he went into business for himself, organizing the firm of Henry C. Meyer Co., dealers in plumbing supplies.

Mr. Meyer's contact with the field of house sanitation so impressed him with the lack of fundamental information on that and allied questions of public health and sanitation that in 1877 he established the *Plumber and Sanitary Engineer*, a monthly magazine for the publication of information on sanitary matters and to serve as a forum for their discussion. Charles F. Wingate, a newspaper man, was appointed editor, and such men as John S. Billings, president of the National Board of Health, Prof. Charles F. Chandler of Columbia University, James J. R. Croes, hydraulic engineer, Prof. Henry Morton, president of Stevens Institute, and Col. George E. Waring, Jr., were called upon as contributors.

Tenement house problem attacked

One of the first problems that Mr. Meyer attacked vigorously through the new paper was the tenement-house condition in New York City. Through the cooperation of sanitary engineers a competition in tenement-house design directed toward elimination of the then existing evils was arranged. The competition received wide attention in the newspapers and resulted in an agitation for an improvement in conditions that culminated in the New York State tenement-house act of 1879.

Similarly, the new journal undertook to improve the plumbing code of New York and Brooklyn, setting forth the fundamental requirements of such a code, and ultimately brought about the adoption of a state law requiring buildings in New York City and Brooklyn to conform to such a code. Mr. Meyer also was active in development of railway car heating by steam to eliminate the hazards of stoves.

In 1881 Mr. Meyer himself assumed charge of the editorial direction of the paper and changed the name to *Sanitary Engineer*. Under his close personal direction its scope was expanded, and in 1887 the name again was changed to the *Engineering and Building Record*, and in 1890 to *Engineering Record*. Mr. Meyer continued in direct charge of the editorial policies of the paper until the late '90s.

In 1902 Mr. Meyer, wishing to retire from active business, sold the paper to James H. McGraw. In 1917 it was combined with *Engineering News* to form *Engineering News-Record*.

Supreme Court Will Hear Nebraska-Wyoming Dispute

The U. S. Supreme Court, on April 1, declined to dismiss the action brought by the state of Nebraska "praying for ascertainment of the equitable apportionment, as between the two states, of the waters of the North Platte River, and a decree to enforce compliance with the findings in that behalf." Wyoming had sought to have the action dismissed on several grounds including those that the state of Colorado and the Secretary of the Interior should be made parties to the action. The court holds that it is not necessary at this time to determine whether Colorado should be a party to the action but that on the face of the bill that state is not a necessary party. As to the Secretary of the Interior, the court states that his rights can rise no higher than those of Wyoming, that Wyoming will stand in judgment for him as for any other appropriator in that state.

The defendant is given 60 days in which to answer the bill of complaint.

BERKELEY BRANCH OF APPROACH TO SAN FRANCISCO BAY BRIDGE

First east approach arm to the San Francisco-Oakland Bridge to be completed is the 1½-mile hydraulic fill along the waterfront north to Ashby Ave., Berkeley. Paving will be the next contract. Other arms will extend east to 38th and Market Sts. and south to 7th and Cypress Sts., Oakland. The north and south arms are also a part of a projected east shore highway to run from San Jose to Albany.

In the foreground of the picture, the new fill turns west to connect with the fill which parallels the Key Route Mole to the East Bay bridge head. All of this fill was constructed by first excavating the soft mud to a depth of 5 to 20 ft. and then pumping in nearly 3,000,000 cu.yd. of sand.

Unemployed Architects to Make New York Zoning Law Survey

As the latest of a series of surveys begun in 1930 to provide work for unemployed architects, the New York Architects Emergency Committee has announced a survey of the zoning classifications of New York City. Since many of the present classifications do not correspond with present conditions, the survey will aid in giving a picture of the living and commercial conditions as they now exist in the city.

Swarthmore College to Hold Convention of Student Chapters

An all-day technical session of the student chapters of eight engineering colleges will be held April 17 at Swarthmore College. The chapters represented will be those of the University of Pennsylvania, Pennsylvania State College, Bucknell, Drexel, University of Delaware, Pennsylvania Military College, Villanova and Swarthmore.

A paper on some current engineering subject will be presented by a representative of each of the chapters. The group will be the guests of the college for luncheon, and the Philadelphia Section of the Am.Soc. C.E. will entertain them for dinner.

National Power Survey Report Shows New Construction Need

(Continued from p. 498)

ditions. As of Jan. 1, 1935, 56 per cent of the total installed steam-electric capacity of the United States was at least 10 years old, 11 per cent was 20 years old or older, and about 1 per cent at least 30 years old. Inefficient and obsolete plants with a capacity of at least 2,000,000 kw. should be scrapped and replaced within the near future.

4. Analysis by districts of the relation between the capacity of existing plants and the demand that will be created with the resumption of normal industrial activity shows that critical shortages will exist in almost every section of the United States. The only regions in which substantial surpluses of capacity now exist to meet normal demand are Florida, parts of Michigan and Illinois, an area along the lower Mississippi, parts of Texas, and North Dakota, Idaho, Utah, New Mexico, Montana, Washington, Minnesota, and Oregon.

5. Government plants provided for or under construction will meet these shortages in certain limited areas. Only one major private power development and one major municipal plant are now under construction. Many of the regions where the greatest power markets now exist and where the shortage will be most acute when industrial activity is renewed do not have projects under construction at the present time sufficient to carry the loads that will develop upon resumption of normal industrial activity.

6. In view of the time required to plan and construct large generating plants, whether hydro or steam, early construction of new plants with an aggregate capacity of between 3,000,000 and 4,000,000 kw. is imperative. This would involve capital expenditures of at least $300,000,000.

Protest Out-of-State Engineers On Los Angeles Flood Control

Protests against the action of C. H. Howell, chief engineer of the Los Angeles, Calif., county flood control district, appointing out-of-state engineers to flood control positions have been filed with the county board of supervisors by Los Angeles county organizations and individuals. Among the objections was a communication from the American Society of Civil Engineers, Los Angeles section, which said in part:

"There are 500 local engineers in our organization duly qualified and capable of holding flood control positions."

Protests also were received by the board from the Association of Land Surveyors of Los Angeles County, the La Crescenta Valley Protective Association and individuals.

Engineer Registration Law Approved for State of Utah

A law calling for the registration of professional engineers and land surveyors, passed at the recent session of the legislature of the State of Utah, has been signed by the Governor and becomes effective at once. The act conforms closely to the model registration law laid down a few years ago by representatives of the major engineering societies of the country. Its administration is placed in the hands of the state department of registration.

The act calls for the appointment by the director of registration of a committee of engineering examiners consisting of seven members, two of whom shall be civil engineers, two electrical engineers, two mechanical engineers, and one a land surveyor. Nominations are to be made from a list of fourteen nominees furnished by local sections of the national engineering societies and the Utah Society of Engineers.

At any time within one year after the act becomes effective, and upon the payment of a registration fee of $5, the director is required to issue a certificate of registration without oral or written examination to any professional engineer or land surveyor who shall submit evidence under oath, satisfactory to the committee of engineering examiners, that he is of good character, has been a resident of the State of Utah and has practiced professional engineering if an engineer, or land surveying if a land surveyor, for at least four years immediately preceding the date of his application and has had responsible charge of work of a character satisfactory to the department.

The bills providing for the registration act were pushed through the two houses of the legislature during the last three days it was in session, the principal opposition coming from mining engineers.

As noted in our issue of March 28, the State of Washington recently enacted an engineer license law specifically exempting mining engineers and members of the American Institute of Mining and Metallurgical Engineers.

Since the passage of the Utah registration act, a movement has been inaugurated to reorganize the Utah Society of Engineers and to form the Utah Society of Professional Engineers to affiliate with the National Society of Professional Engineers.

A registration law has been passed by the New Mexico state legislature but the details are not yet available.

$55,000,000 Bond Issue Proposed For New York State Work

The New York State Legislature has been asked in a special message by Governor Lehman to pass a bond issue proposal which would provide $55,000,000 in ten-year unemployment relief bonds, $20,000,000 of which would be used for "the construction of permanent necessary public works," while the balance would be used to reimburse local governments for the state's share of relief activities during the year 1936.

The bond issues previously authorized will all be about used up at the end of the present year.

It is expected that all of the proposed $20,000,000 would be used for enlargement and extension of the state's hospital and prison facilities. The population in institutions has been growing rapidly during the past few years, while during the same period the state has drastically curtailed its normal program of construction of all permanent improvements.

The state's total capital outlay appropriation for public improvements has dropped from $44,400,000 in 1932 to $17,800,000 in 1933, and to $11,700,000 for 1935.

The proposed bond issue will have to be passed on by vote of the people at the next general election, following approval of it by the legislature.

SOCIETY CALENDAR

NATIONAL RIVERS AND HARBORS CONGRESS, annual convention, Washington, D. C., May 2 and 3.

AMERICAN WATER WORKS ASSOCIATION, annual convention, Cincinnati, Ohio, May 6-10.

AMERICAN SOCIETY FOR TESTING MATERIALS, annual meeting, Detroit, June 24-28.

AMERICAN SOCIETY OF CIVIL ENGINEERS, annual convention, Los Angeles, July 3-5.

SOUTHERN OHIO SOCIETY OF PROFESSIONAL ENGINEERS, composed of civil engineers from Fayette, Ross, Highland, Pike, Brown, Adams and Clinton counties, has been organized. Frank W. Turner, Fayette county surveyor, has been elected president. Other officers elected are Edwin F. Beace, Ross, vice-president; C. L. Sapp, Clinton, second vice-president; E. C. Zimmerman, Adams, treasurer, and William Boulware, Highland, secretary.

THE ENGINEERS' SOCIETY OF WESTERN PENNSYLVANIA, Civil Section, devoted the afternoon and evening of March 5 to a conference on monumental bridge foundations. At the afternoon session a paper entitled "Design and Construction of the Major Foundations of the San Francisco-Oakland Bay Bridge" by C. H. Purcell, chief engineer, and Charles E. Andrew, principal bridge engineer, was presented, and at the evening session a paper entitled "The Foundations of the Mississippi Bridge at New Orleans" by F. M. Masters, consulting engineer, Modjeski, Masters & Case, was presented. The two papers were discussed by Wilson Ballard, vice president, J. E. Greiner Co.; M. F. Clements, bridge engineer, Northern Pacific Railway; and J. S. Miller, president, Dravo Contracting Co. Informal discussion was directed toward the three methods of construction employed, namely, the sand-island method for the New Orleans bridge, and the dome method for the West bay, and the open cofferdam method for the East Bay portion of the San Francisco-Oakland Bay bridge.

ENGINEERING REGISTRATION EXAMINATIONS are scheduled to be held as follows:

WISCONSIN—State Capitol, Madison, Wis.; Civil Engineers, Apr. 24-26.

Personals

CASIMIR J. PLAGENS, of Detroit, Mich., has resigned his position of secretary of the city Board of Water Commissioners. He has accepted a similar position with the Detroit Lighting Commission.

EDWARD STINGEL, of New Philadelphia, Ohio, former county surveyor and state highway engineer, has been appointed a resident county engineer for the state highway department.

DANIEL J. BRUMLEY, chief engineer for fourteen years of the Illinois Central Railroad and an employee of the road since 1903, has been placed on the retirement roll, having attained the 70-year age limit. Mr. Brumley entered railroad service in 1895 following his graduation that year from Ohio State University. He is a past president of the Western Society of Engineers and has held civic office as village president in his home of Flossmoor. He is now a member of a park board.

DUGALD C. JACKSON, head of the electrical engineering department at the Massachusetts Institute of Technology since 1907, past president of the Society for the Promotion of Engineering Education, and for many years senior partner of the consulting firm of Jackson & Moreland, Boston, will retire from his post at the institute at the end of the current academic year. Professor Jackson's varied career as an engineer includes professional association with many important transportation and hydroelectric projects in this country and elsewhere, and many engineering appraisals in the public utility field.

Obituary

SAMUEL J. KELLEY, for 50 years a contractor at Cambridge, Mass., died at his home in that city on March 26 at the age of 97. He was a native of Yarmouth, N. S., and retired from business 25 years ago.

O. E. NOBLE, civil engineer of Manhattan, Kan., died on March 8 at the age of 60. Mr. Noble was a graduate of Kansas State College and for many years was active in municipal engineering in Kansas and Oklahoma.

CONSTRUCTION STATISTICS OF THE WEEK

HIGHER public awards raised this week's engineering construction total to $25,156,000 compared with $18,584,000 last week and the average week of April, 1934 of $26,602,000. Of this total, $3,511,000 represents private awards and $21,645,000 public awards. Of the public work, $4,463,000 is federal and $17,182,000 state and municipal.

The largest increase in awards for the week is in the public buildings classification which is 280 per cent higher than last week and, at $8,588,000, is the highest week this year. Waterworks at $2,573,000 is a gain over last week, as is sewerage at $1,551,000. Bridges dropped to the low volume of $453,000 while streets and roads increased slightly to $4,162,000. Both industrial and commercial buildings are lower than last week, the former at $843,000 and the latter at $1,530,000.

The larger awards for the week include Marina Jr. High School, San Francisco, Calif., $566,000; Public School 43, New York City, $748,000, high school, Tottenville, S. I., N. Y., $771,000; University Housing Project, F. E. H. Corp., Atlanta, Ga., $1,984,000. The larger highway lettings include $364,000 by Tennessee; and $430,000 by New York. Reservoirs and coagulation basins for waterworks filtration plant, Milwaukee, Wisc., were awarded at $741,000. At Los Angeles, synchronous condensers for Department of Water and Power were awarded at $732,000 and transformers at $793,000, and at Chicago, the Chicago-Northwestern Ry. awarded 25,000 tons of steel rails and 9,000 tons of fastenings at approximately $950,000.

New capital for the week includes $15,374,000 in state and municipal bond sales, of which $10,000,000 represents new financing by the Port of New York Authority. The total new capital outstanding was decreased by net reductions in PWA allotments outstanding of $12,630,000, to total excess of rescissions and reductions this week over new allotments and increases in outstanding allotments.

CONTRACTS

(Thousands of Dollars)

	Weekly Apr. 1934	Average Prev. 4 Weeks	Week Apr. 4 1935
Federal Government	$6,389	$3,373	$4,463
State and municipal	15,432	13,845	17,182
Total public	$21,821	$17,218	$21,645
Total private	4,781	5,521	3,511
Week's total	$26,602	$22,739	$25,156
Cumulative to date:			
1934...$404,587,000	1935...$332,467,000		

NEW PRODUCTIVE CAPITAL

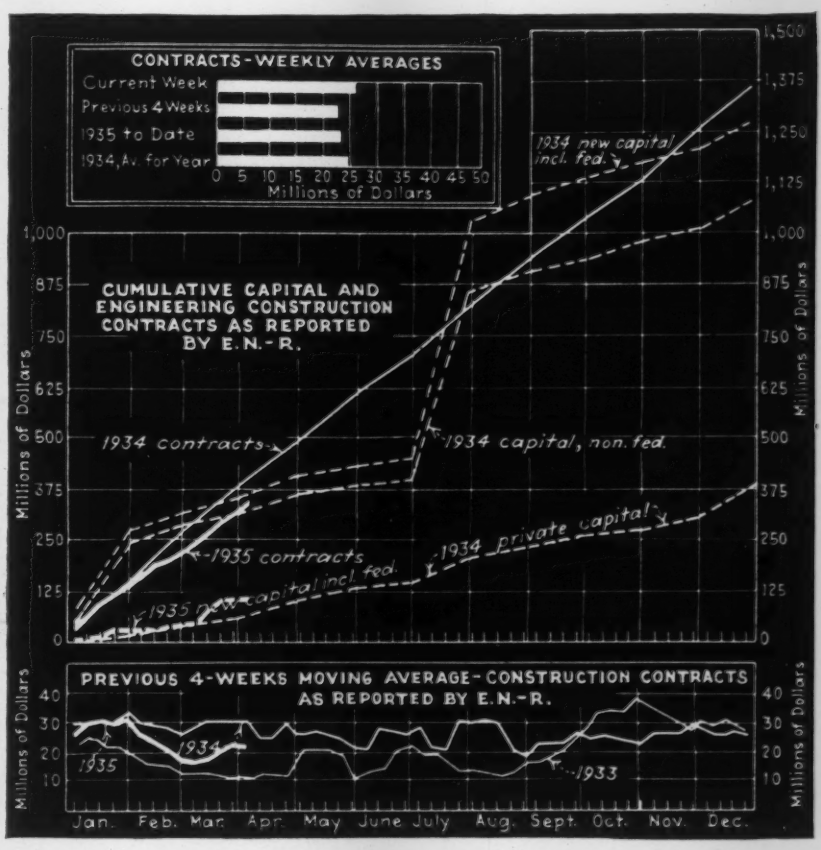
(Thousands of Dollars)

1935	Week Apr. 4	Cumulative
State and municipal	$15,374	$125,740
PWA allotment, S&M	—11,546	—44,950
RFC loans, S&M	....	84
Corporate issues	....	....
PWA allotments, private	—150	7,129
Total, Non-Federal	$3,678	$88,033
PWA allotments, Federal Constr.	—934	7,336
Total new capital	$2,744	$95,369
Cumulative to date:		
1934...$361,069,000	1935...$95,369,000	

Note: These figures include private bonds, and stocks sold for productive purposes; state and municipal bonds for construction; PWA loans and grants to states and municipalities, including the special highway funds; PWA private loans, and allotments for Federal construction. Negative amounts in PWA totals indicate excess of rescissions and reductions over reallotments and increases in outstanding allotments.

INDEX NUMBER

E.N.R.-Cost	1913 = 100	1926 = 100	E.N.R.-Volume	1913 = 100	1926 = 100
Apr., 1935	194.50	93.49	Mar., 1935	101	44
Mar., 1935	194.26	93.38	Feb., 1935	76	33
Apr., 1934	195.86	93.90	Mar., 1934	116	51
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.16	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56



BUSINESS SIDE OF CONSTRUCTION

March Construction Volume Higher Middle Atlantic Territory Gains

IN March, engineering construction awards increased to a total of $90,958,000 for the four weeks. Private works make up $22,084,000 of this total and public works $68,874,000, of which $13,493,000 is federal construction. While these totals are lower than March of last year, they represent a larger number of medium sized and smaller projects, since the March, 1934, total was dominated by large federal work in the South, and the Ford steel mills in the industrial building group. For the first time this year, private awards total less than the corresponding month last year, but private work continues ahead of last year on the total volume to date since the first of the year. Calls for bids continue to run steadily higher than a year ago, averaging 227 compared with 201 per week in March, 1934.

Highway awards in March increased slightly to $19,568,000, but continue below the volume of awards last year. Public building awards are up over 50 per cent from February and are slightly higher than March a year ago. Industrial buildings show a substantial increase over February, but are below the high volume of March a year ago that was due to the Ford steel plant.

The Middle Atlantic territory increased its gain over last year, and to date is the only territory running ahead of a year ago. New England contract awards are running neck to neck with last year, while awards in the South show the effect of not having large federal awards to duplicate those of last year. The Middle West has dropped slightly below a year ago due to lower volume in March. West of Mississippi, which had a high February peak a year ago due to the Fort Peck dam, is running below last year, as is also the Far West territory. All territories except the South show gain in March over last month.

New Capital

A substantial gain in new capital was recorded for March due to the sale of $53,657,000 in state and municipal bonds. The Public Works Administration rescinded or decreased outstanding allotments in excess of new allotments to the extent of $1,200,000 which represents a reduction in the total amount of PWA allotments outstanding.

Materials

Cement—Cement shipments in February totaled 2,962,000 bbl. as reported by the U. S. Bureau of Mines. This represents a slight increase over January and is approximately the same as the shipments in February, 1934. The ratio of production to capacity declined from 20.2 in February, 1934, to 14.9 in February, 1935. The February ratio, however, was an increase from 14.1, the January, 1935, ratio.

Steel—The operating rate of steel companies having 98.7 per cent of the steel capacity of the industry increased the first week of

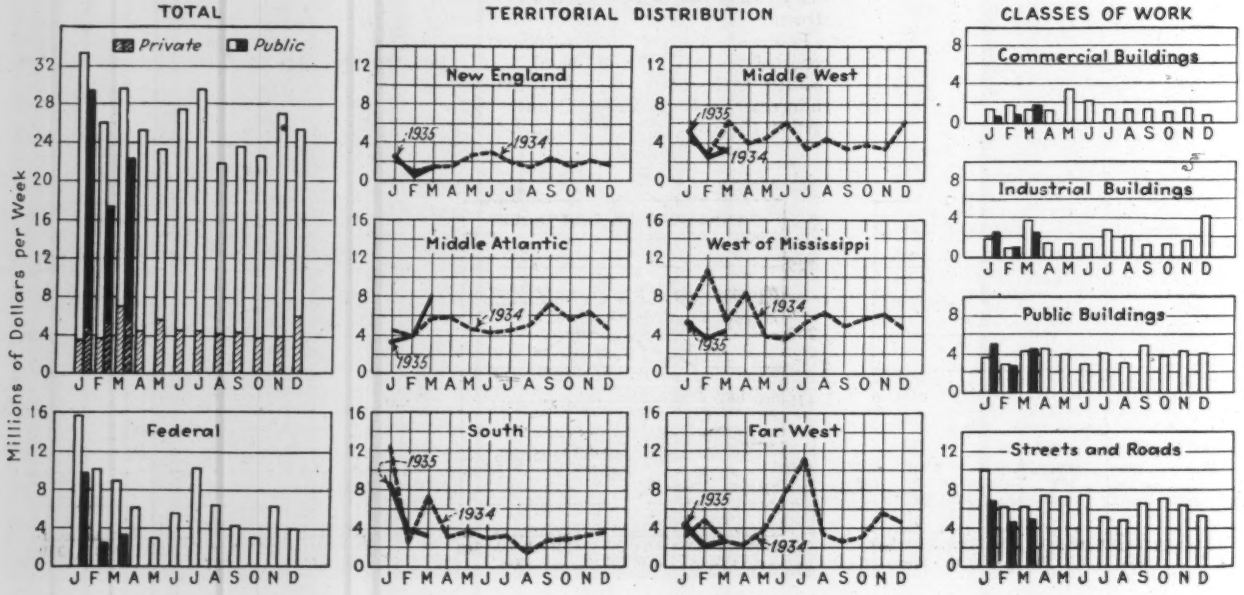
ENGINEERING CONSTRUCTION CONTRACTS REPORTED IN MARCH, 1935

Four Weeks—Thousands of Dollars—(000 Omitted)

	New England	Middle Atlantic	South	Middle West	West of Mississippi	Far West	United States March 1935	United States Three Months 1935	United States Three Months 1934	Canada
Public Works										
Waterworks	415	667	576	275	360	1,711	4,004	17,127	13,166	1,250
Sewerage	173	2,161	182	842	435	23	3,816	14,180	10,736	37
Bridges, public	145	3,931	477	1,798	1,002	839	8,192	20,661	26,057	.....
Earthwork and waterways	174	76	3,532	1,411	2,458	504	8,155	51,029	113,941	.....
Streets and Roads	1,497	2,125	4,297	4,243	5,315	2,091	19,568	72,058	96,088	656
Buildings, public	2,389	4,920	2,072	979	5,289	2,172	17,821	55,074	48,598	3,455
Unclassified, public	172	6,234	21	480	147	264	7,318	11,134	12,499	176
Total public	4,965	20,114	11,157	10,028	15,006	7,604	68,874	241,263	321,085	5,574
Federal gov't (included in above classifications)	1,148	1,006	4,611	2,215	2,990	1,523	13,493	71,194	147,908	.....
Private										
Bridges, private	.....	1,070	25	.....	.....	50	1,145	4,581	1,866	.....
Buildings, industrial	384	4,417	308	2,030	552	923	8,614	24,423	30,091	140
Buildings, commercial	180	4,258	.....	326	1,644	491	6,899	14,283	18,444	.....
Unclassified, private	205	2,242	1,246	198	277	1,258	5,426	22,761	13,560	.....
Total private	769	11,987	1,579	2,554	2,473	2,722	22,084	66,048	63,961	140
March, 1935 (4 weeks)	**5,734**	**32,101**	**12,736**	**12,582**	**17,479**	**10,326**	**90,958**	.....	.....	**5,714**
February, 1935 (4 weeks)	2,304	16,048	15,586	9,007	15,302	9,842	68,089	.....	.....	5,049
March, 1934 (5 weeks)	7,157	29,238	37,887	31,364	27,915	13,643	147,204	.....	.....	6,074
Three months, 1935	20,344	65,950	70,958	47,955	60,376	41,728	.....	307,311	.....	14,794
Three months, 1934	20,105	63,029	97,751	60,593	98,219	45,349	.....	.....	385,046	7,831

ENGINEERING CONSTRUCTION REPORTED BY E.N.-R.—WEEKLY AVERAGES

□---1934 ■—1935



March to 48.2 per cent of the steel capacity of the industry, but declined during the succeeding three weeks to 46.1 per cent for the last week of March. These rates are consistently higher than corresponding rates a year ago, which showed a similar decline in the same period. These statistics are reported by the American Iron and Steel Institute.

Fabricated structural steel bookings during February approximated in volume the business done during the same month last year, according to report of the American Institute of Steel Construction, based on data representing 80 per cent of the industry. The large drop in bookings during January keeps the record of business to date this year under last year's average. Shipments during February were ten per cent less than the average for the first six months of 1934. The back log is approximately 34 per cent less than at the same time last year.

Lumber—Lumber shipments reached this year's highest volume in the second week of March when 198,000 ft. was shipped, representing a gain of 22 per cent over the same week last year. Shipments averaged approximately 177,600,000 ft. per week during March as reported by the National Lumber Manufacturers' Association. New business booked was consistently ahead of last year and was running ahead of production reported during the last of the month.

Building Permits

Building permits reported monthly by Dun & Bradstreet totaled $27,651,002 for February which is 42.1 per cent ahead of a year ago, and a 4 per cent increase over January, 1935. The gain over February, 1934, was smallest in the South Atlantic Territory, 4.3 per cent, but was substantial in other territories ranging from 33.3 per cent in the Middle Atlantic to 82.9 per cent in the South Central. The gain in New York City was 95.4 and the average gain outside of New York City was 33.5 per cent. These percentages are based on reports from 215 cities.

The Bureau of Labor Statistics, U. S. Department of Labor, reports a gain in building permits for February, 1935, over February, 1934, of 32.2 per cent in estimated cost and 39.7 per cent in number, based on reports from 775 identical cities having a population of 10,000 or over.

The gain on new residential work was 99.3 per cent in estimated cost, and on new non-residential work 11.2 per cent, while the gain in additions, alterations and repairs was 30.4 per cent in estimated cost.

Labor

The employment index for the production of six major construction materials as reported by the Federal Employment Stabilization Office is 37.9 for February, 1935, compared with 38.0 for January, 1934, and 37.9 for February, 1934. The base of this index is 1923-25 = 100.

Employment on public and private building construction projects not aided by Public Works Administration funds declined 12 per cent in January from the December total, based on reports from 9,963 firms to the U. S. Bureau of Labor Statistics. In individual cities the decline in employment on building ranged from 1.1 per cent in Youngstown, Ohio, to 52.7 per cent in Tacoma, Wash., and increased employment was reported in Atlanta, Ga., 6.8 per cent, Fort Wayne, Ind., 0.5, South Bend, 18.6, Des Moines, Ia., 8.7, Wichita, Kan., 17.6, Louisville, Ky., 5.1, Baltimore, Md., 4.8, Grand Rapids, Mich., 16.9, El Paso, Tex., 5.0, and San Antonio, 22.2 per cent.

Nearly 305,000 men were employed at the site of Public Works Administration projects during the month of January 15, a decline of 78,000 compared with December. Of this total, 191,829 wage earners were employed on federal projects and 97,957 on non-federal work.

Employment reported by the Department of Labor in the State of New York represted 1,041 firms engaged in construction projects. Compared with December, January employment dropped 17 per cent, payrolls 15 per cent, and man hours 17 per cent. Decreases for the corresponding period a year ago amounted to 15 per cent in employment, 9 per cent in payrolls and 11 per cent in man hours. The greatest decrease occurred in highway construction groups where employment dropped 42 per cent, while on general building, employment dropped 13 per cent, and 13 per cent for sub-contracting, while miscellaneous general contracting dropped 8 per cent. The 219 general building contractors reporting employed 6,115 workers this month compared with 7,058 in December. Highway contractors in January dropped about 1,750, or 42 per cent in December's employees.

Industrial fatalities reported by the New York State Department of Labor for February charge only 4 workers' deaths to construction.

Few wage and jurisdictional disputes are reported for March. On April 1, housepainters in Essex County, N. J., will seek an increase from the Master Painters' Association from $1 an hour for an 8-hr. day and 5-day week, to $1.28½ an hour, or $9 a day for a 7-hour day, with double rate for overtime under both schedules.

In Oklahoma City non-union carpenters have been invited to meet with the Carpenters' Union to consider wage and hour provisions for a regional code.

In Rochester organized labor placed before the City Manager its demand for skilled labor for skilled work in city service. About a dozen city workmen are affected due to the practice of having city employees do the work of skilled workmen in conjunction with other duties.

A jurisdictional dispute between two labor unions stopped work on the new Internal Revenue Bureau addition in Washington. Both the hoisting engineers and the elevator constructors claim they are entitled to the job of hoisting materials for the construction of elevators, and each is backed by their own international union. About 300 workmen are idle for lack of material with which to work. The plumbers who had materials on hand were the only tradesmen who worked.

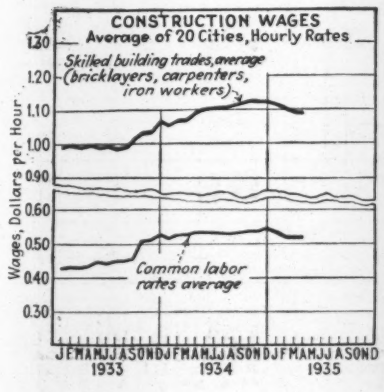
In Salt Lake City, Utah, relief project workers walked out on a strike asking for $32 a month for each family of two, with $12 for each additional person, with these sums stabilized. Under the present system social workers determine the amount for individual families according to their needs. They also ask for a minimum wage rate of 65 cents per hour instead of the present rate of 50 cents, transportation to and from project, and time to be allowed during transportation, increased rate for teamsters, all foremen to be selected from relief rolls; all workers to be placed on projects as close to their residences as possible, and a 30-hour week instead of 24.

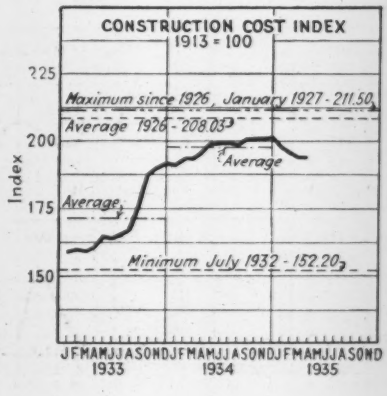
EN-R Cost and Volume Index

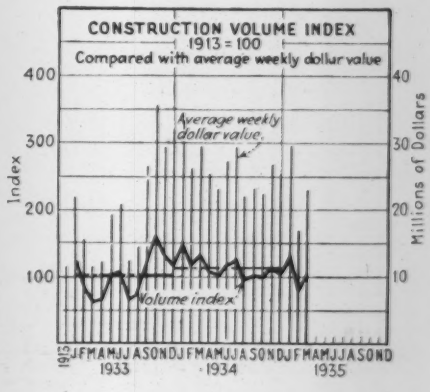
The Engineering News-Record Construction Cost Index for April is 194.50 compared with 194.60, the corrected figure for March. The change is due to a drop in lumber, small increase in cement, and a slight increase in labor.

The Volume Index for March is 101 compared with 76 last month, and 130 a year ago when the index was increased 22 points by a single large federal project.

	INDEX NUMBER				
E.N.R.-Cost	1913 = 100	1926 = 100	E.N.R.-Volume	1913 = 100	1926 = 100
Apr., 1935	**194.50**	**93.49**	**Mar., 1935**	**101**	**44**
Mar., 1935	194.26	93.38	Feb., 1935	76	33
Apr., 1934	195.86	93.90	Mar., 1934	116	51
1934 (Av.)	198.10	95.23	1934 (Av.)	114	50
1933 (Av.)	170.18	81.80	1933 (Av.)	102	47
1932 (Av.)	156.97	75.45	1932 (Av.)	127	56


CONSTRUCTION WAGES
Average of 20 Cities, Hourly Rates
Skilled building trades, average (bricklayers, carpenters, iron workers)
Common labor rates average
Wages, Dollars per Hour
1.30
1.20
1.10
1.00
0.90
0.60
0.50
0.40
0.30
J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
1933
1934
1935


CONSTRUCTION COST INDEX
1913 = 100
Maximum since 1926, January 1927 - 211.50
Average 1926 - 208.03
Average
Average
Minimum July 1932 - 152.20
Index
225
200
175
150
J F M A M J J A S O N D J F M A M J J A S O N D J F M A M J J A S O N D
1933
1934
1935



CURRENT PRICES OF CONSTRUCTION MATERIALS

CEMENT, AGGREGATES AND READY-MIXED CONCRETE—F.O.B. CITY

	PORTLAND CEMENT — Per bbl., carload lots, including 40c. per bbl. for bags, cash discount not included			SAND AND GRAVEL — Per ton, carload lots			CRUSHED STONE — Per ton, carload lots		CRUSHED SLAG — Per ton, carload lots, f.o.b. plant		CONCRETE — Ready Mixed 1:2:4, 50c.y. or more, delivered
	Cloth Bags*	Paper Bags	Bulk	Gravel, 1½ in.	Gravel, ¾ in.	Sand	1½ in.	¾ in.	1½ in.	¾ in.	
Atlanta	$2.89	$2.64	$2.44	$2.31	$2.31	$1.64	$1.69	$1.92	$1.80	$1.80	$8.25
Baltimore	2.66	2.41	2.21	1.80‡	2.00‡	1.25‡	1.80‡	1.80‡	2.25	2.25	7.35
Birmingham	2.55	2.30	2.10	1.75	1.75	1.40	.85	.85	.90	1.15	5.45
Boston	2.72	2.47	2.27	1.60	1.60	1.15	1.60	1.60	.....	.....	7.00
Chicago	2.95‡	2.70‡	2.40	2.60‡	2.60‡	2.05‡	2.60‡	2.60‡	.....	.....	.....
Cincinnati	2.57	2.32	2.12	1.20	1.20	1.10	1.80	1.80	.....	.....	6.70
Cleveland	2.45	2.20	.....	1.55	1.55	1.45	1.60	1.60	1.75	1.75	8.85
Dallas	2.45	2.20	2.00	1.55	1.55	1.30	2.10	2.25	.....	.....	7.00
Denver	3.08	2.83	.....	1.85	.....	1.25	.....	.....	.....	.....	.....
Detroit	2.25	2.00	1.80	1.40	1.30	1.20	1.10	1.10	1.30‡	1.30‡	6.50
Kansas City	2.50	2.25	2.05	.....	1.85	1.15	1.85	1.40-1.55	.....	.....	7.65
Los Angeles	2.70	2.42	.....	1.60	1.60	1.20	1.60	1.60	.....	.....	6.70
Minneapolis	2.85	.....	2.40	1.00[p]	1.00[p]	.25[p]	1.00[p]	1.00[p]	.....	.....	6.50
Montreal	.....	1.70	.....	.85	.90	.40	.85	.95§	.....	.....	6.00
New Orleans	2.37	2.15	2.00	1.85	1.85	1.50	.....	.....	.....	.....	7.65
New York	2.60‡	2.35‡	.....	1.30†[k]	1.30†[k]	.85†[k]	1.75†[k]	1.85†[k]	.....	.....	7.60
Philadelphia	2.60	2.35	.....	1.80‡	1.90‡	1.50‡	1.75‡	2.00‡	1.75‡	1.75‡	7.75
Pittsburgh	2.45	2.20	1.60	1.50‡	1.50‡	1.70‡	2.25‡	2.25‡	1.50	1.50	7.00
St. Louis	2.42	2.17	.....	1.25‡	1.25‡	1.25‡	.75[p]	.75[p]	.75§	.75§	6.65
San Francisco	2.72	2.52	2.20	1.35	1.35	1.35	1.50	1.50	.....	.....	7.53
Seattle	2.95	2.70	.....	1.00	1.00	1.00	1.44	1.44	.....	.....	7.56

*10c. allowed for each returnable bag. †Per cu. yd. ‡Delivered. §F.o.b. Granite City, Ill. [k]Barge lots alongside dock. [p]f.o.b. plant. [s]5 tons or more.

CURRENT MAXIMUM NET PRICES RECEIVED AT CEMENT MILL

Charge for bags not included. For cloth bags, add 40c. per bbl.; 10c. refund allowed for each returnable bag; for paper bags add 15c. per bbl., not refundable.

	Bagged	Bulk		Bagged	Bulk		Bagged	Bulk
Alpena, Mich.	$1.70	$1.65	Ironton, Ohio	$1.60	$1.55	North Birmingham, Ala.	$1.75	$1.70
Buffington, Ind.	1.60	1.55	Iola, Kans.	1.70	1.65	Richard City, Tenn.	1.85	1.80
Crestmore, Calif.	1.72	.....	La Salle, Ill.	1.60	1.55	Saginaw, Mich.	1.50	1.45
Dallas, Tex. (Inc. 5c. tax)	1.80	1.75	Limedale, Ind.	1.70	1.65	Steelton, Minn.	1.80	1.75
Hannibal, Mo.	1.70	1.65	Mason City, Ia.	1.70	1.65	Universal, Pa.	1.65	1.60
Hudson, N. Y.	1.75	1.70	Norfolk, Va.	1.76	1.71	Waco, Tex. (Inc. 5c. Tax)	1.80	1.75
Independence, Kans.	1.70	1.65	Northampton, Pa.	1.65	1.60	Wyandotte, Mich.	1.45	1.40

Cash Discounts, Cement, to Contractors: truck delivery, 2% for cash on 10th of month; on carload deliveries, no trucking, same as dealer cash discount, 10c. per bbl. for payment within 15 days of date of invoice.

STRUCTURAL CLAY BUILDING TILE, BRICK AND LIME—DELIVERED

	STRUCTURAL CLAY TILE—PARTITION—SCORED — Per M, lots of 2,000 pieces or over,			STRUCTURAL CLAY TILE—LOAD BEARING—SCORED — Per M, lots of 2,000 pieces or over,			BRICK — Per M, in quantity,		LIME —		
	3x12x12 in.	4x12x12 in.	8x12x12 in.	8x12x12 in.	10x12x12 in.	12x12x12 in.	Common backing	Straight hard	Per ton, Hydrated finishing	in paper, Common hydrated	carload lot, Pulverized or lump
Atlanta	$75.80	$80.80	$151.60	$181.90	$212.20	$242.50	$14.25	$14.25	$24.00	$17.00	$15.90
Baltimore	74.50	80.00	150.00	180.00	209.00	260.00	13.00	17.00	16.75	11.25	19.80
Birmingham	84.25	90.00	156.00	180.00	215.00	242.00	15.00	22.50	3.00§	1.75§	2.25§[s]
Boston	85.30	91.15	171.00	193.50	239.60	273.40	14.50	18.00	19.00	14.00	3.36[g][s]
Chicago	66.00	70.40	132.00	.....	.....	.....	10.20	11.20	18.00	13.40	14.00
Cincinnati	69.95	74.60	139.90	139.90	174.65	199.20	17.50	17.50	15.97	12.48	.....
Cleveland	62.50	66.50	124.50	149.50	188.00	214.75	16.50	16.50	17.20k	14.40k	2.65k[p]
Dallas	72.00	75.00	197.00	177.00	217.00	269.00	13.50	26.00	21.20	14.50	1.70 [p]
Denver	78.50	88.00	141.50	166.50	196.00	225.00	10.00-15.50	21.00-30.00	25.00	17.00	20.00
Detroit	63.60	67.80	127.10	230.00	270.00	330.00	11.50	12.50	14.30	12.05	21.56
Kansas City	72.50	75.50	92.00*	140.50	177.75	207.25	15.00	24.00	15.00	11.25	14.00[s]
Los Angeles	77.50	94.25	134.00*	188.00	232.50	311.50	11.50	14.00	18.70	.....	19.70
Minneapolis	80.15s	85.50s	145.40s	167.75s	175.25s	243.70s	13.10	15.10x	25.50	19.00	21.00
Montreal	79.10s	90.40	169.50s	.....	212.40	.....	17.00	22.50	23.00	13.00	.....
New Orleans	61.60	65.70	123.20	158.90	196.70	.....	14.00	.....	16.75	13.05	.....
New York	84.00	90.00	135.00*	193.30†	238.80†	295.60†	11.50	.....	20.80	16.00	20.80
Philadelphia	80.50	85.90	161.00	182.50	235.00	272.00	15.00	18.00	16.35	11.25	10.75
Pittsburgh	72.65c	77.45c	145.00c	165.00c	217.00c	250.00c	17.50	22.00	16.70	15.10	2.75[p]
St. Louis	62.00	65.00	120.00	165.00	187.00	214.00	15.00	18.00	.48‡	.38‡	2.80[p]
San Francisco	84.00	94.50	225.00	.....	.....	.....	15.00	18.00	22.50	21.50	2.05[p]
Seattle	86.00	94.50	210.00	198.00	.....	.....	15.00[d]	18.50	30.00	26.00	3.00[p]

*6x12x12 in. †F.o.b. Perth Amboy, N. J. ‡Per sack. §Per bbl., 200 lbs. [p]Per bbl. 180 lbs. [g]per bbl. 280-lb. kLCL. [d]less $1 cash 15 days. [s]lump. [r]Smooth. x Selected common.

ROAD SURFACING MATERIALS—F.O.B. CITY

	PAVING BRICK AND BLOCKS — Granite per M, lots of 50,000, 4x4x8 in.,	Brick per M, 3x8½x4 in., carload lots	Wood per sq.yd., 3½ in., 16-lb. treat,	PAVING ASPHALT — Per ton, less than 80 penetration, Tank car	Drums	ASPHALT BINDERS—FLUXES — Per gal., 80-300 penetration, Tank car	Drums	CUTBACK ASPHALT — Per ton, Tank car	Drums	ROAD OILS — Per gal., Tank car	ASPHALT EMULSION — Per gal., Tank car	Drum
Atlanta	$ 80.00	$35.00	$2.25	$17.40[z]	$22.47[z]	$0.0694[z]	$0.961[z]	$19.65[z]	$25.57[z]	$0.0694[z]	$0.105	$0.105
Baltimore	110.00	44.00	2.25	17.00	22.00	.07	.095	.085†	.12†	.....	.....	.....
Birmingham	110.00	24.00	.....	18.00	23.00	0.778	.1028	.082†	.112†	.....	.06	.....
Boston	85.00	30.00	2.75	16.00	21.00	.075	.10	.09†	.12†	.075	.09	.125
Chicago	138.00	42.00	2.60	.....	.....	.....	.....	.....	.....	.....	.....	.....
Cincinnati	115.00	37.00	.....	18.00	22.00	.09	.....	18.50	.....	.07	.08	.....
Cleveland	100.00‡	32.75	2.95	20.00	24.50	.075	.09	.075†	.049†	.06	.085	.10
Dallas	.....	30.00	.....	16.10	21.10	.0525	.10	15.65	23.15	.055	.1076	.1438
Detroit	.....	37.00	.....	19.67[z]	24.87[z]	.....	.....	17.00	22.50	.....	.....	.....
Kansas City	.....	45.00	.....	18.30	23.30	17.30#	22.30#	.075†	.105†	.055	.11	.15
Los Angeles	.....	50.00	.....	10.34	14.50	10.34#	14.50#	10.34	.....	.042	.06	.12
Minneapolis	.....	.....	2.50	18.10	24.60	.098	.098	21.75	34.55	.048	.....	.....
Montreal	.....	110.00	.....	14.00	19.66	.07	.093	19.00	26.00	.07	.125	.155
New Orleans	100.00	32.00	.....	14.50	18.50	.065	.09	.07†	.10†	.....	.....	.....
New York	110.00§	50.00	2.64	17.00	22.00	.07	.095	.085†	.12†	.....	.085	.12
Philadelphia	110.00	45.00	2.25	15.00	20.00	.06	.....	.07†	.10†	.06	.....	.....
Pittsburgh	120.00	40.00	.....	19.00	24.00	.0775	.1125	.086†	.121†	.0675	.....	.....
St. Louis	117.50	35.00	2.00	19.50	24.50	.078	.108	.088†	.118†	.04	.09	.125
San Francisco	.....	50.00	.....	12.00	18.00	12.00#	18.00#	12.50#	23.00#	.04	.0575	.117
Seattle	.....	55.00	.....	17.00	23.00	17.00#	23.00#	15.00	.....	.03	.19	.20

Note: Paving asphalt, tank car or boat, f.o.b. Maurer, N. J., per ton, Bermudes, $25.00; Trinidad, $21.00. *Per sq. yd. †Per gallon. ‡4½x6x5½in. #Per ton §4x5x9-in. Av. [z]Mexican. [z]local reduction due to 20% reduction intra-state class freight rates, only Georgia affected.

CURRENT PRICES OF CONSTRUCTION MATERIALS

IRON AND STEEL PRODUCTS—BASE MILL PRICES

	STRUCT. SHAPES-PLATE	REINF. BARS ¼-in. billet,	RIVETS ½-in. structural	WIRE NAILS Base	SHEET PILING Base	STEEL RAILS Per Gross Ton Standard	Light	Re-rolled	TRACK SUPPLIES Angle bars	Std. spikes	Tie plates	Track bolts
Birmingham	$1.95	$2.10	.....	.....	.....	$36.375	$34.375	$35.00	$2.55	$2.40	$1.90	$3.55
Chicago	1.85	2.10	$3.00	.....	$2.25	36.375	35.00	34.00	2.55	2.40	1.90	3.55
Pittsburgh	1.80	2.05	2.90	$2.60	2.15	36.375	35.00	34.00	2.55	2.40	1.90	3.55

IRON AND STEEL PRODUCTS—BASE MILL PRICES, PER 100 LB., CARLOAD LOTS

F.O.B. WAREHOUSE	STRUCTURAL SHAPES Per 100 lb., base price	REINFORCING BARS Per 100 lb., base price New billet	Rail steel	EXPANDED METAL LATH Per 100 sq. yd., carload lots Std. diamond mesh, 3.4 lb.	Std. ribbed 3.4 lb.	WELDED FABRIC REINFORCING Per 100 s.f., carload lots 4x16 in., No. 5 & 10 wires	4x12 in., No. 8 & 12 wires	6x6 in., No. 6 & 6 wires Per sq. yd	SHEET PILING Per 100 lb., base price
Atlanta	$3.34	$2.32	$2.17	$21.00	$24.00	$1.53	$1.16	$0.1449	$2.65
Baltimore	2.95	2.75	2.60	22.00	25.00	1.41	1.08	.1332	2.75
Boston	3.52	3.03	2.2125	20.50	24.50	1.46	1.12	.1449	2.30
Chicago	3.20	2.10	1.95	22.00	26.00	1.40	1.08	.1323	2.25
Cincinnati	3.40	3.25	3.25	25.01	29.50	1.39	1.07	.1287	....
Cleveland	3.16	2.60	2.45	20.00	22.00	1.37	1.06	.1287	....
Dallas	3.25	2.80	2.65	22.00	25.00	1.63	1.23	.1548	2.985
Denver	3.91	3.76	....	25.00	27.50	1.68	1.26	.1548	....
Detroit	3.40	2.885	2.885	20.20†	23.60†	1.41	1.08	.1323	2.25
Kansas City	3.53	2.68	2.53	19.00	22.00	1.50	1.14	.1422	2.67
Los Angeles	3.10	2.95	....	23.00	27.50	1.68	1.26	.1584	2.50
Minneapolis	3.45	2.82	2.67	22.00	26.00	1.51	1.15	.1431	2.55§
Montreal	2.75	3.00	2.80	....	....	2.30	1.71	.238	2.65
New Orleans	....	....	....	19.15	23.00	1.54	1.17	.1458	....
New York	3.37	2.82	2.67	20.00	22.00	1.43	1.10	.1350	2.48
Philadelphia	2.005	2.34	2.19	20.00	23.50	1.42	1.09	.1332	2.44
Pittsburgh	1.80‡	1.80‡	1.70‡	20.75‡	23.00‡	1.34	1.04	.1260	2.15
St. Louis	3.44	2.44	2.29	24.00	25.00	1.42	1.09	.1341	2.25
San Francisco	3.55	2.475	2.35	23.00	28.00	1.68	1.26	.1584	2.60
Seattle	3.50	2.50	....	30.00	33.00	1.68	1.26	.1584	2.60

‡Base Mill Price. Note: Shapes, bars and piling subject to quantity discounts. §Mill Price plus freight to Minneapolis. † .o.b. delivered.

PAINTING AND ROOFING SUPPLIES—F.O.B.

	RED LEAD Per 100 lb. in 500-lb. lots Dry	WHITE LEAD Per 100 lb. in 500-lb. lots# in oil	READY-MIXED PAINT Per gal., drums Graphite*	Aluminum†	Ferric Oxide‡	ROOFING SUPPLIES Carload lots, f.o.b. factory Rolls, slate surfaced, 85-90 lb., per sq.	Asphalt felt, per 100 lb.	Tar felt, per 100 lb.	Asphalt coating, per gal	Tar pitch 350-lb. bbl. per ton
Atlanta	$7.75	$10.50	$1.60	$3.10	.....	$1.81	$1.97	$2.90	.36	$25.40
Baltimore	7.00	10.00	1.40	2.25	$1.90	2.32	2.55	2.55	.48	24.60
Birmingham	7.25	10.50	2.35	2.35	1.85	1.95	1.79	....	36	23.00
Boston	7.00	10.00	1.85	2.75	.04‡x	2.10	3.10	3.10	.45	20.00
Chicago	7.00	10.00	1.85	2.45	1.30	1.60	1.59*	1.59*	.24	20.00
Cincinnati	7.00	10.00	1.40	3.00	1.55	2.40	2.39*	2.39*	.36	21.00
Cleveland	7.00	10.00	1.65	2.30	1.30	1.60	1.79	1.79*	.24	22.00
Dallas	7.25	11.00	1.80	2.24	.....	2.40‡	4.17	4.17	.15	35.00
Denver	7.50	10.75	2.55	.....	.....	2.75	3.46	3.46	.60	30.00
Detroit	7.00	10.00	1.45	2.20	1.30	2.55	2.80	2.80	.37	30.00
Kansas City	7.00	10.12½	1.25	2.25	1.00	2.05	3.10–3.20	3.10–3.20	.23	26.50
Los Angeles	7.25	10.25	1.60	2.25	2.30	2.00	1.50	.....	.45	.....
Minneapolis	7.00	10.12½	....	....	....	1.60	1.79*	1.79*	.29	25.10
Montreal	....	9.40	1.70	3.45	1.65	2.00	2.62	2.02	1.40§	1.52§
New York	7.00	10.00	1.65	2.20	1.45	1.60	1.79*	1.79*	.232	22.00
Philadelphia	7.00	10.00	1.50	2.05	1.50	1.68	1.85*	1.85*	.34	22.00
Pittsburgh	7.00	10.00	1.05	1.95	.80	1.75	1.65*	1.65	.38	22.00
St. Louis	11.00	11.00	1.85	2.45	1.30	1.87½	3.22	3.22	.28	20.00
San Francisco	7.25	9.75	1.80	2.75	1.10–2.00	1.89	3.10	3.10	.38	26.00
Seattle	7.25	9.75	1.80	2.85	2.25	3.30	2.66	2.60	.60	30.00

*U. S. War Dept Spec. 3-49A. †ASTM Spec. D266-31. ‡80% maximum ferric oxide. §Per 100 lb. ▲Per roll, 65 lb. xPer lb. #500-lb. drums.
Note: Red lead in oil 50c higher than white lead in oil. #Basis of quoting changed this month from list to 500-lb. lots.

MISCELLANEOUS ITEMS

WINDOW GLASS

Discounts from latest jobbers list, Sept. 15, 1928

	Single or Double Thickness A quality	B quality
Atlanta	85%	85%
Baltimore	88–10%	90–10%
Birmingham	88%	90%
Boston	88–10–5%	90–10–5%
Chicago	90%	92%
Cincinnati	89–10%	91–10%
Cleveland	89-10%	91-10%
Dallas	87%	89%
Denver	84%	86%
Detroit	87–10%	89–10%
Kansas City	88–10%	90–10%
Los Angeles	90%*	90–40%
Minneapolis	85%	87%
Montreal	.....	.....
New Orleans	88%	90%
New York	88%	90%
Philadelphia	88–30%	90–30%
Pittsburgh	88%	88%
St. Louis	90%	92%
San Francisco	90%	90–30%
Seattle	90–10%	90–20%

*Disc. from list Aug. 1, 1929.

EXPLOSIVES

Per lb. dynamite delivered 50-lb. cases

	Gelatin 40%	60%
Atlanta	$0.155	$0.17
Baltimore	.16	.175
Birmingham	.105	.120
Boston	.17	.1725
Chicago	....	....
Cincinnati	.27	.29
Cleveland	.19	.205
Dallas	.17	.185
Denver	.155	.17
Detroit	.237	.255
Kansas City	.16	.175
Los Angeles	.1725†	.1925†
Minneapolis	.17	.185
Montreal	.1875	.2125
New Orleans	.19	.22
New York	.225	.24
Philadelphia	....	....
Pittsburgh	.11	.1250
St. Louis	.135	.15
San Francisco	.1575	.1775
Seattle	.1425	.1575

†F.o.b. Arsenal. ‡Double thickness A & B.

CHEMICALS

Water, sewage treatment, road work, f.o.b., carlots, New York.	
Bleaching powder, in drums, f.o.b. works, per 100 lb.	$1.90@$2.15
Calcium chloride, 77-80%, flaked, in 400-lb. drums or 100-lb. moisture proof bags, f.o.b. works, freight equalised with points of competition, per ton	22.00@35.00
Chlorine cylinders, per lb. delivered	.05½
Silicate of soda, 52 deg. in drums, f.o.b. works, per 100 lb.	1.35
Soda ash, 58%, in paper bags, per 100 lb	1.10
Sulphate of aluminum, commercial, in 100 lb. bags per ton	27.00
Sulphate of copper, in bbl., per 100 lb.	3.85

PILES

Prices per linear foot, pine, with bark on f.o.b., New York; delivered from barge, 1½ to 2c. per ft. additional:

Dimensions	Points	Length	Short Leaf Barge	Short Leaf Rail	Long Leaf Barge	Long Leaf Rail
12 in. at butt	6 in.	30 to 50 ft.	$0.12	$0.165	$0.13	$0.16
12 in.—2 ft. from butt	6 in.	50 to 59 ft.	.16	.21	.15	.20
12 in.—2 ft. from butt	6 in.	60 to 69 ft.	.17	.225	.16	.21½
14 in.—2 ft. from butt	6 in.	50 to 69 ft.	.195	.235	.20½	.24½
14 in.—2 ft. from butt	6 in.	70 to 79 ft.	.2125	.245	.24½	.30½
14 in.—2 ft. from butt	5 in.	80 to 85 ft.	.275	.385	.....	.....
14 in.—2 ft. from butt	5 in.	85 to 89 ft	.315	.452	.....	.....

RAILWAY TIES

Prices f.o.b., per tie, for carload lots:

		6 In. x 8 In. by 8 Ft.	7 In. x 9 In. by 8½ Ft.
Boston	Oak, untreated	$1.15	$1.60
	Pine, untreated	1.15	1.60
	Pine, creosoted	1.60	2.25
New York	Long-leaf sap pine, untreated	.80	1.00
	Mixed oak, untreated	.....	1.00
	White oak, untreated	.75	1.50
Birmingham	White oak, creosoted	no market	
	Southern pine, untreated *6x8x8'6"	.45*	.60
	Southern pine, creosoted	1.00*	1.20
Chicago	White oak, untreated	1.15	1.40
	Oak, empty cell, creosoted	1.50	1.88
	Oak, sine treated	1.40	1.70
	Southern pine, creosoted	1.35	1.78
Los Angeles	Fir, untreated	.50	.72
	Fir, creosoted	1.67	2.40
Philadelphia	Red oak, untreated	1.05	1.35
	Red oak, creosoted	1.50	1.90
	White oak, untreated	1.10	1.45
St. Louis	Red oak, untreated	1.08	1.40
	Red oak, creosoted	1.45	1.95
	Sap Pine or cypress, untreated	.83	1.25
San Francisco	Douglas fir, green, untreated	.45	.63
	Douglas fir, empty cell, creosoted	1.14	1.56
Montreal	Birch or maple, untreated	1.00	1.10
	Birch or maple, creosoted	1.25	1.55

CURRENT PRICES OF CONSTRUCTION MATERIALS

WATER, SEWER AND DRAIN PIPE

	C. I. PIPE	VITRIFIED SEWER PIPE (‡ f.o.b.)				CLAY DRAIN TILE		CONCRETE SEWER PIPE		WROUGHT STEEL PIPE			
	Per net ton, f.o.b. 6 in. to 24 in.*	Per foot, delivered, standard 8 in., s.s.	12 in., s.s.	24 in., d.s.	36 in., d.s.	Per 1,000 ft., carload lots, f.o.b. 6 in.	8 in.	Per ft., delivered; ASTM C 14-24 12 in.	24 in.	Full standard weight. # 1 to 3 in.; Black	Butt Weld Galv.	3½ to 6 in., Black	Lap Weld Galv.
Atlanta	$42.00	$0.225	$0.405	$1.4625	$3.25	$80.00	$128.00	$0.55	$1.60	44%	37%	42%	34%
Baltimore	45.50	.24	.46	2.05	5.65	105.00	165.00	.75	2.00				
Birmingham	38.00	.25	.45	1.625	4.66	85.00	200.00	.45	1.60	48.939%	37.918%	50.162%	39.143%
Boston	45.50	.231	.4455	2.025	5.945	60.10	101.00	.54	1.90				
Chicago	46.00	.28	.54	1.98	5.60	85.00	120.00	.675	2.25	56%	45%	55%	37%
Cincinnati	45.00	.231	.4455	1.935	5.8425	69.00	108.00	.40	1.40	53.04%	42.02%	54.265%	43.245%
Cleveland	46.50	.21	.405	1.71	5.2275	47.25	72.00	.536	2.102	58.4%	48.4%	60.6%	49.6%
Dallas	50.00	.25	.45	1.80	.....	160.00	250.00	.41	1.41	64%	55%	64%	56%
Denver	58.00	.23‡	.415‡	1.73‡	.....	.....	.....	.....	.....	56%	45%	43%‡	57%‡
Detroit	46.50	.238	.456	1.935	5.74	58.10	97.20	.35	1.55	48.6%	38.2%	47.3%	37.6%
Kansas City	48.00	.32	.57	2.27	4.90	120.00	240.00	.35	1.20	58%	49%	59%	50%
Los Angeles	46.50	.2825‡	.508‡	2.03‡	5.535‡	115.50	192.50	1.00	2.00	53.57%	43%	53%	42.51%
Minneapolis	48.50	.283	.51	2.04	5.13	.....	.....	.50	1.33	.....	.....	.....	.....
Montreal	.....	.80	1.50	6.00	.....	45.00	90.00	.45	.....	.....	.....	.....	.....
New Orleans	44.00	.282	.465	1.65	.....	60.00	.....	.51	1.69	.....	.....	.....	.....
New York	45.00	.26	.51	2.25	6.75	.....	.....	.55	2.00	51.88%*	40.86%*	53.10%*	42.08%*
Philadelphia	42.00	.315	.6075	2.70	.....	150.00	230.00	.....	.....	52.37%*	41.35%*	53.39%*	42.57%*
Pittsburgh	48.00	.224‡	.432‡	1.845‡	5.535‡	69.00	108.00	.75	1.75	64%	55%	65%	56%
St. Louis	45.80	.22	.396	1.584	3.96	85.00	172.00	.80	2.28	51.51%	40.49%	52.74%	41.71%
San Francisco	46.00	.265	.477	1.908	5.22	105.00	175.00	.45	2.20	52.4%	63.4%	51.1%	64.6%
Seattle	47.50	.34†	.60†	2.40†	6.30†	85.00	200.00	1.50	1.60	60.3%	51.3%	65.3%	52.3%

*b. & s. class B and heavier, C/L lots, 200 tons and over. Burlington, N. J. (base) $42.00. Gas pipe and class A, $3 per ton additional. 4-in., $3 per ton additional. 30 in. and larger usually $2 per ton less. ‡Price at factory. †Less 30% in C/L Lots.

#Discounts from standard list for delivery from warehouse, except Pittsburgh prices are f.o.b. mill. Base price $200 per net ton. List prices per ft.: ¼ in., 8½c; ⅜ in. 11½c.; 1 in., 17c.; 2 in., 37c.; 2½ in., 58½c.; 3 in., 76½c.; 4 in., $1.09; 6 in., $1.92.

‡2½ to 3 in. *Resale price to consumer, carload lots shipped from mill.

LUMBER AND TIMBER—PER M FT. B.M., CARLOAD LOTS F.O.B.

		SHORT LEAF YELLOW PINE AND DOUGLAS FIR — All S. L. Y. P. is No. 2 common or better and for No. 1 N. C. Box. All Fir planks No. 2 common; Fir timber is No. 1 common. Lengths up to 20 ft.									LONG LEAF Y. P. Merchantable grade up to 20 ft.	
		1x6,S	1x8,S	2x4,S	2x6,S	2x8,S	2x10,S	3x12,R	6x12,R	12x12,R	2x12,R	12x12,R
Atlanta	Southern Pine	$16.00	$17.00	$19.00	$16.50	$18.00	$18.50	$30.00	.....	.....	$38.50	$40.00
Baltimore	Southern Pine	23.00	23.00	28.50	28.00	28.60	30.50	37.00	.....	.....	54.00	55.00
	Douglas Fir	28.00	28.00	30.00	28.00	30.00	30.50	30.00	$30.00	$30.00		
Birmingham	Long Leaf	13.25	14.25	16.25	14.25	15.25	15.25	37.00			27.00	34.80
	Southern Pine	13.25	14.25	15.00	13.00	14.00	14.00	24.00	24.00	23.00	.....	.....
Boston	Southern Pine	32.00	33.00	40.00†	42.00†	55.00‡	64.00‡	76.00‡			76.00	76.00
	Douglas Fir	31.00	32.00	40.00	40.00	40.00	42.00	43.00	45.00	45.00	.....	.....
Chicago	Southern Pine	27.00	28.00	33.50	30.00	31.00	32.80	45.00			54.00	58.00
	Douglas Fir	28.50	29.50	31.50	30.50	30.50	28.50	40.75	41.75	41.75	.....	.....
Cincinnati	Southern Pine	26.00	27.00	27.50	25.50	26.50	27.50	43.50			43.50	.....
	Douglas Fir							46.25	46.25	46.25		
Cleveland	Southern Pine	26.00	27.00	31.00	27.00	28.00	30.00	43.00	43.00	.....	46.00	55.00§
Dallas	Southern Pine	d28.35	30.60	31.50	29.80	30.15	30.15	50.40	.....	.....	42.75	67.95
Denver	Douglas Fir	38.00	39.00*=	40.00*	40.00*	40.00*	42.00*	49.00*	53.00*	58.50		
Detroit	Southern Pine	26.00	27.50	30.00	27.00	30.00	32.00	37.00			32.00	43.75
	Douglas Fir							43.25	44.25	44.25		
Kansas City	Southern Pine	22.50	23.50	23.50	22.50	23.50	24.50	35.00			47.50	57.50
	Douglas Fir	22.75	23.75	26.00	25.00	25.00	26.50	35.00	35.00	35.00	.....	.....
Los Angeles	Douglas Fir	24.00„	25.50„	26.00„	26.00„	26.00„	26.00„	38.00„	38.00„	34.00„	.....	.....
Minneapolis	Western Pine	43.00*	41.00*	34.00*	31.50*	32.00*	33.30*	46.00*k			46.00*	.....
	Douglas Fir	30.00*	32.50*	33.50*	31.50*	32.00*	33.00*	45.00*	46.00*	46.00k	.....	.....
Montreal	Spruce	30.00	31.00	27.00	29.00	30.00	35.00	38.00	.....	.....	.....	.....
	Douglas Fir	42.50	42.50	42.50	44.00	44.00	45.75	51.00	51.00	51.00	43.00	.....
New Orleans	Southern Pine	27.90	28.95	33.70	28.90	31.35	32.40	43.95	.....	.....	45.65	52.45
	Douglas Fir	28.95	31.50	33.90	29.35	32.90	33.95	37.95	.....	.....	.....	.....
New York	Southern Pine	32.00*	33.00*	32.00*	33.00*	34.00*	35.00*	45.00*			70.00*	70.00*
	Douglas Fir	.....	.....	36.00*	35.00*	36.00*	37.00*	36.00*	39.00*	39.00*	.....	.....
Philadelphia	Southern Pine	24.50	25.50	27.00	25.50	26.50	27.00	31.00			61.00	59.00
	Douglas Fir	.....	.....	31.00	31.00	31.00	31.00	32.50	32.50	33.50	.....	.....
Pittsburgh	Southern Pine	26.50	27.75	32.75	29.25	31.75	32.75	41.50			57.50	57.50
	Douglas Fir	33.50	34.50	34.50	33.50	34.00	34.25	46.25	46.25	46.25	.....	.....
St. Louis	Southern Pine	25.00*	26.00*	28.00*	27.00*	28.00*	29.00*	55.00*			56.00*	67.00*
	Douglas Fir	.....	.....	.....	.....	.....	.....	55.00*	55.00*	55.00*	.....	.....
San Francisco	Douglas Fir	15.00t	16.00t	17.00t	14.50t	15.50t	16.50t	21.00t	21.00t	21.00t	.....	.....
Seattle	Douglas Fir	13.00	14.00	14.00	11.50	12.50	12.00	18.50	18.00	18.00	.....	.....

*Delivered. †Spruce. ‡Long-leafed Y. P. §Norway Pine. ‖Northern Pine. =Native. tAt ship's tackle. „Yard prices. d10% Disc taken off.
aS4S. kRgh.

CURRENT BUILDING AND CONSTRUCTION WAGE RATES PER HOUR

	Bricklayers	Carpenters	Structural Iron Workers	Hoisting Engineers	Plasterers	Common Labor — Building	Common Labor — Heavy Const.
Atlanta	$0.70/1.125	$0.40/.90	$0.60/1.25	$0.60/1.25	$0.60/1.00	$0.30/.40	$0.30/.40
Baltimore	1.25	1.10	1.37½	1.25	1.25	.45	.40
Birmingham	1.00	.60/1.00	1.00	1.125	1.00	.30/.40†	.30/.40†
Boston	1.20	.70/1.17½	1.20	.90/1.37½	1.37½	.40/.70	.40/.70
Cincinnati	1.27½/1.375	1.02½/1.20	1.20/1.25	1.07½/1.25	1.27½/1.375	.45	.45
Chicago	1.50	1.31¼	1.35	1.31¼	1.50	.82½	.82½
Cleveland	1.25	1.12½	1.25	1.25	1.25	.72	.72
Dallas	1.00	1.00	1.00	1.00	1.00	.40	.40
Denver	.60/1.12½	.60/.87½	.60/.87½	.60/.87½	.....	.50	.50
Detroit	1.25	1.00	1.25	.75/.95	1.25	.50	.50/.70
Kansas City	1.32½	1.12½	1.125	1.12½	1.32½	.70	.45/.50
Los Angeles	1.10	1.10	1.125	1.10	1.25	.50	.60
Minneapolis	.90/1.00	.75	.90	.75	.90	.40	.50
New Orleans	1.00	.75	1.00	1.00	1.25	.30/.40	.30/.40
New York	1.50	1.40	1.65	1.50/1.65	1.50	.82½	.50/.82½
Philadelphia	.75/1.20	.60/1.20	1.00/1.20	.75/1.20	.60/1.20	.30/.50	.30/.50
Pittsburgh	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.50/1.00	.35/.50	.35/.50
St. Louis	1.50	1.25	1.47	1.35/1.47	1.50	.78¾	.35/.78
San Francisco	1.50*	1.10	1.37½	1.12½	1.25*	.62½	.60
Seattle	.90/1.00	.90/1.00	1.00/1.30	1.00/1.25	.90/1.50	.45/.65	.50/.65
Montreal	1.00	.70	.80	.55	.80	.40	.25

Skilled Avg: (Bricklayers, Carpenters, Ironworkers) $1.095. Common Avg. $0.5255 *6-hr. Day. †PWA.